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USE OF MANUFACTURED SANDS FOR CONCRETE PAVING

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by

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Dissertation

Presented to the Faculty of the Graduate School of

The University of Texas at Austin

in Partial Fulfillment

of the Requirements

for the Degree of

DOCTOR OF PHILOSOPHY

The University of Texas at Austin

August 2011

Acknowledgements

I would like to convey my sincere appreciation to my supervisor, Dr. David Fowler, for his kindness, trust, and for giving me the opportunity to work on this project. Dr. Fowler was always receptive and supportive through all the stages of my research. I would also like to thank Dr. Maria Juenger, Dr. Raissa Ferron, Dr. Amit Bhasin, and Dr. Harovel Wheat of my dissertation committee for their valuable time and suggestions.

I would like to thank the Texas Department of Transportation for funding this project. The assistance provided by the project director, Ryan Barborak, and the project advisors, Dr. German Claros, Elizabeth Lukefahr, Caroline Herrera, Edward Morgan, and Ralph Browne was also invaluable. Their interest, suggestions, and continuous support were essential in completing this project.

Special thanks are due to David Whitney without whom this project would not have been successful. David Whitney was always willing to listen, encourage, and provide technical and moral support whenever I needed it. Michael Rung's dedication in helping me on my project is also greatly appreciated; he was extremely helpful whenever I needed his assistance.

The work done on this project by John Clement was crucial for completing this research on time, since he spent a lot of his valuable time repairing and upgrading my testing equipment. I believe that the graduate students at the Construction and Materials and Research Group are lucky to have a student as kind and helpful as John Clement amongst them. My friend Michael De Moya was available to help me with my work when I mostly needed help. Michael's assistance on this project and moral support will not be forgotten. I would like to thank Sarwar Siddiqui for helping me with aggregate testing.

I would also like to thank all the undergraduate students that worked with me during those last three years. The help provided by Karla Kruse, Efren Tavira, Claudia Patterson, Pilar Guerrero, and Nicole Rockey was vital in completing all the physical work that was required. They helped do work that I could not have possibly done by myself. I enjoyed working with each one them and I wish them all a successful career.

Finally, I wish to express my gratitude and appreciation to my parents for their continuous love, support, and concern. My parents have always been there for me and have supported every decision I have made in every way they could. Leaving them to pursue my education in the United States was one of the hardest decisions I made.

Use of Manufactured Sands for Concrete Pavement

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The University of Texas at Austin, 2011

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Manufactured fine aggregates are a product created when rocks are crushed using a mechanical crusher. With the depletion of sources of natural sands, the usage of manufactured fine aggregates has increased. Manufactured fine aggregates have properties that differ from natural sands; for this reason, the plastic and hardened properties of concrete produced using manufactured fine aggregates differ from the properties of concrete made with natural sands. The main concrete properties affected by the usage of manufactured fine aggregates are skid resistance, workability, and finishability.

The aim of this research project was to investigate how manufactured fine aggregates could be used in concrete pavements without causing workability or skid related issues. To improve the workability of concrete made with manufactured fine aggregates, the use of the optimized mixture proportioning method developed by the International Center for Aggregate Research (ICAR) was investigated. Results obtained from this testing were used to make recommendations on how the ICAR method for pavement concrete could be improved

The goal of this research was to also develop laboratory tests that could reasonably predict skid performance of concrete pavements made with different types of sand. For this purpose concrete slabs made with different sands were evaluated for friction and texture using a circular texture meter (CTM), a dynamic friction tester (DFT), and a polisher. To ensure that the values obtained at the laboratory related to field performance, test sections constructed with 100% limestone sand and blended sands were evaluated. Laboratory and field test results for skid were used to identify aggregate tests that best correlates with concrete performance. Results show that the micro-Deval test for fine aggregates could be used to predict the polish resistance of concrete laboratory specimen. Results from field testing has shown that if limestone fine aggregates are not blended with siliceous sands, PCC pavements made with limestone sands on truck lanes could experience a large drop in skid resistance within a year of service. Results obtained from laboratory testing showed that blending a small quantity of siliceous sand with limestone sands considerably increased the skid resistance of concrete specimens.

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Chapter 1: Introduction

1.1 BACKGROUND

Sources of quality natural sands have begun depleting in metropolitan areas where the need for concrete is high. In such areas the concrete industry has the option to either ship natural sands from outside sources or use local sources of manufactured fine aggregates (MFA). Shipping aggregates from outside sources adds to the cost of concrete, and it is important to find methods to maximize the use of local materials.

Several problems arise from using MFAs in class pavement concrete including workability, finishability, and skid resistance. These problems exist because of the mineralogy, shape, or grading of MFAs. In general, MFAs are less polish resistant than natural sands. An increase in surface polishing leads to a decrease in skid resistance and potentially higher incidences of skid-related accidents on highways. Skid resistance depends on the surface macro-texture and micro-texture. In PCC pavements the long-term skid resistance is a function of the type of fine aggregate. Softer sands like carbonate sands are believed to provide less long-term skid resistance when compared with harder siliceous sands. No recent research has been done to evaluate skid resistance of PCC made with limestone sands, and thus it is not clear whether or not current specifications adopted by state agencies accurately reflect the performance of those sands in the field

Workability and finishability problems exist as a result of the poor shape and grading of many MFAs. To overcome the poor shape and grading of MFAs, additional paste is added to the mixture; the addition of more paste adds to the cost of concrete and affects its durability.

To improve the workability and finishability of mixtures made with MFA, higher paste content is required. Using higher paste content increases the cost of making concrete and it also reduces the durability of concrete.

1.2 PROBLEM STATEMENT

Many state agencies like the Texas Department of Transportation (TxDOT) have set limits on the usage of carbonate sands. In Texas, the current limits are determined by the acid insoluble test residue (AIR) test that has a minimum required value of 60% for sands used in PCC pavements. Under the current specifications, the maximum quantity of carbonate sand that can be used in a PCC pavement is less than 40% of the total sand volume since the carbonate sands generally have an AIR value of less than 10%. The Dallas and Ft. Worth districts have limited local sources of natural siliceous sands but many sources of manufactured carbonate sands (mostly limestone). Since most of the local sources of manufactured fine aggregates do not meet the specifications, those districts have to transport aggregates that meet the specifications from distant pits (which increases cost). One of the problems with the acid insoluble residue test is that it is a chemical test, while polishing is a mechanical phenomenon. For this reason it was important to investigate if manufactured sands could be used in concrete without affecting skid resistance.

Another concern in using MFA in PCC pavements involves workability and finishability. Compared to natural sands, concrete made with MFA yields less workable and finishable mixtures for the same mixture proportions. In 2008, three sections containing 100% manufactured sands were constructed as part of an implementation project in the Fort Worth district. Major workability and finishability problems were encountered during the construction of those sections. The concrete made with 100% MFA did not meet the workability requirements for slip-form concrete; the mixtures were either too harsh or too workable. The mixture design used for that implementation project was a mixture design typically used for blended sands.

1.3 RESEARCH OBJECTIVES

The ultimate aim of this research project was to examine how more manufactured sands could be used in PCC pavements without affecting the quality of the concrete produced. To achieve this objective several issues needed to be addressed, including:

- Finding improved aggregate tests that could be used by the TxDOT to accept fine aggregates for PCC pavements (fine aggregate tests that relate to skid resistance).
- Investigating whether or not modern crushing operations could improve the shape of manufactured sands.
- Finding a better proportioning method for designing PCC pavement mixtures.
- Investigating field sections made with fine aggregates that do not meet the acid insoluble residue limits
- Developing a laboratory concrete test to evaluate the skid resistance of concrete.
- Evaluating laboratory specimens made with different fine aggregates and aggregate blends.
- Investigating if a change in mixture proportions could improve the skid resistance of pavement made with MFA.

Chapter 2: Aggregates in Concrete Literature Review

Natural sand has been almost exclusively used in pavement concrete. As the sources of natural sands are diminishing, manufactured sands have been considered as an alternative. Manufactured fine aggregates (MFA) are produced by crushing quarried stones into smaller sized aggregates. These aggregates have properties different from the natural aggregates that have been historically used. These differences in properties have lead to problems involving proportioning of mixtures and the ability to obtain the fresh and hardened properties required for paving. It has also been alleged that carbonate MFA polish more, resulting in lower surface friction. This literature review discusses the properties of aggregates that affect concrete performance: shape, texture, grading, and mineralogy. Other topics relevant to this dissertation are also discussed in this chapter: methods of crushing aggregate, blended sands, and approaches used for optimizing aggregate gradation.

2.1 AGGREGATE PROPERTIES

2.1.1 Shape

The shape of the aggregate particles influences paste demand, placement characteristics such as workability and pumpability, strength, and cost [O'Flynn, 2000]. Shape is related to sphericity, form, angularity, and roundness.

- The sphericity measures how nearly equal are the three principal axis of the aggregate (length *L*, width *W*, and height *H*). The sphericity increases as the three dimensions approach equal values [Brzezicki and Kasperkiewicz, 1999; Graves, 2006].

- The form or the shape factor, describes the relative proportions of the three axes of a particle. It helps distinguish between particles that have the same sphericity [Graves, 2006].
- The angularity describes the proportions of the average radius of curvature of corners and edges to the radius of maximum inscribed circle [Graves, 2006].
- The roundness describes the sharpness of the edges and corners [Graves, 2006].

Particle shape can be classified by the following descriptions:

- *Sphericity & form*: cubical, spherical, flat or elongated. [Graves, 2006; Brzezicki and Kasperkiewicz, 1999].
- *Angularity & roundness:* Angular, subangular, subrounded, rounded, well-rounded. [Graves, 2006; Brzezicki and Kasperkiewicz, 1999].

The descriptions of angularity and roundness are illustrated in Figure 2.1 and detailed here:

- *Angular*: little evidence of wear on the particle surface
- *Subangular*: evidence of some wear, but faces untouched
- *Subrounded*: considerable wear, faces reduced in area
- *Rounded*: faces almost gone
- *Well rounded*: no original faces left

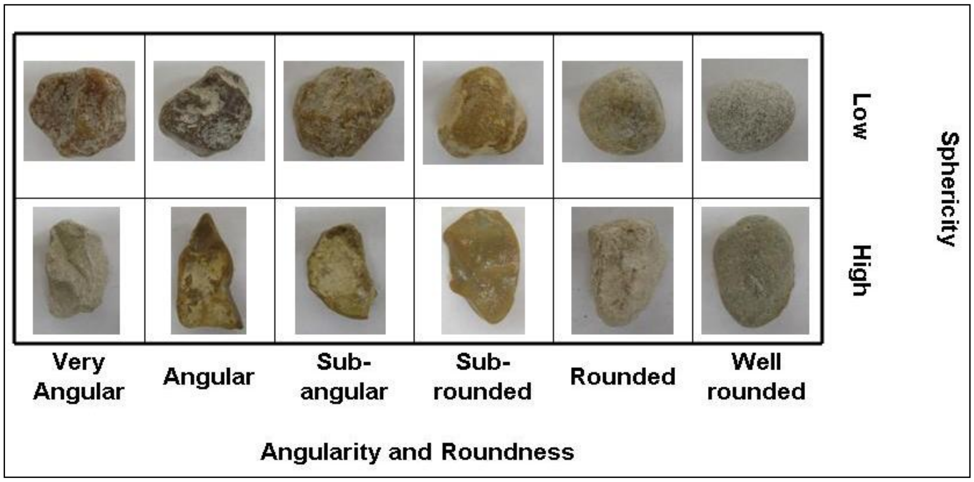

Figure 2.1: Particle Shape

Round or nearly cubical shaped aggregates are desirable due to the ease in which they move in the mixing and handling process. However, aggregates can also contain flat or elongated shapes. Methods used to measure the shape of coarse aggregates are the elongation factor and flatness factor. A flat particle has a width-to-thickness ratio greater than or equal to 3, while an elongated particle has a length-to-width ratio greater or equal to 3. Specifications usually define limiting elongation ratios of 3:1 or 5:1 to describe undesirable shapes of aggregates. The shape can modify the strength of the concrete, as in the case where a thin, flat particle is oriented in the hardened concrete where outside stresses are introduced [Graves, 2006].

The shape of natural aggregates depends on the strength, abrasion resistance, and on the degree of wear to which they have been subjected in their depositional environment. Natural aggregates tend be more spherical and less angular. On the other hand, the shape of manufactured aggregate depends on the rock type (mineralogy) and the crushing equipment. Manufactured aggregates are more angular when compared to natural aggregates [Graves, 2006].

The shape of an aggregate influences the workability of the mixture as well as the void content and packing density. For the same amount of paste, a mixture with round or cubical-shaped aggregate will have better workability than a mixture with flaky and elongated aggregates. Moreover, for the same mass of aggregates, round and cubical aggregates produce mixtures with higher packing, which results in a lower void content [Fowler et al., 2008]. The decreased percentage of voids lowers the amount of cement paste required for that particular mixture. Some specifications, such as the Spanish and British standards [Quiroga and Fowler, 2004], limit the percent of use of flaky and elongated particles, but ASTM (American Society for Testing and Materials) has set no limits. Some state departments of transportation (DOTs) have set limits on the percentage of flaky and elongated particles ranging from 8 to 20%.

The shape of fine aggregates affects concrete workability more than the shape of coarse aggregates [Fowler et al., 2008]. Since fine aggregates are smaller in size than coarse aggregates, a larger volume of paste is needed to coat the fine aggregates. When poorly-shaped fine aggregates are used, the paste requirement to achieve the target workability becomes substantial [Fowler et al., 2008]. This is one of the main reasons that poorly-shaped fine aggregates are not desirable in concrete. Unlike coarse aggregates, the shape of fine aggregates is not always directly evaluated. Indirect methods have been used to evaluate the shape of fine aggregate; such methods include ASTM D 3398 (standard method for Index of Aggregate of Particle Shape and Texture) and ASTM C 1252 (Standard Test Method for Uncompacted Void Content of Fine aggregate as Influenced by Particle Shape, Surface Texture, and Grading). Both methods evaluate shape indirectly by measuring the packing density of a re-graded fine aggregate sample. Aggregates with better shape such as natural siliceous sands are expected to have higher packing density than the poorly-shaped manufactured sands. Electronic equipment has also been used to evaluate aggregate shape. One of the more widely used equipment for evaluating shape is the Aggregate Imaging Measurement System (AIMS). AIMS captures and analyzes images of multiple particles and is capable of directly evaluating the form and angularity of fine aggregates. AIMS evaluates the shape of fine aggregates by using a 2D form index that ranges from 1 to 20. The lower the form index the more equidimensional a particle is. AIMS also evaluates the angularity of fine aggregates. The scale used ranges from 0 to 10000; 0 indicates the presence of well round aggregates, and 10000 indicates the presence of highly angular aggregates.

2.1.2 Texture

Surface texture is the degree to which the surface may be defined as either: 1) being rough or smooth (referring to the height of asperities) or 2) coarse grained or fine grained (referring to the spacing between grains) [Graves, 2006]. The surface texture influences the workability, quantity of cement and bond between particles and the cement paste. Two independent geometric properties are the roughness or rugosity (degree of surface relief) and the roughness factor (the amount of surface area per unit of dimensional or projected area) [Graves, 2006].

Natural aggregates have a smooth surface [Graves, 2006]. Natural gravel subject to transport mechanisms tends to be smoother than manufactured aggregates. For instance, gravel would have a surface smoother than crushed limestone. An improvement in the bond to the matrix is obtained as the surface roughness increases [Ahn and Fowler, 2001]. Rough-textured angular grains bond better with the cement paste to generate higher tensile strengths [O'Flynn, 2000]. Although rougher textures lead to better bond between paste and aggregate, they also lead to harsher mixtures, as texture roughness increases, the internal friction increases between the aggregates, and therefore more paste is needed to achieve a given workability. There are no direct methods to measuring the texture of fine aggregates. ASTM D 3398 and ASTM C 1252 can be used to indirectly evaluate texture of fine aggregates (as well as shape).

2.1.3 Grading

The gradation of an aggregate is defined as the frequency of a distribution of the particle sizes of a particular aggregate [Graves, 2006]. Grading limits are specified in ASTM C 33 section 6 [ASTM C 33]. For state jobs in Texas, aggregate grading has to meet the TxDOT Standard Specifications for Construction and Maintenance of Highways, Streets, and Bridges item 421 requirements. Aggregate grading can be divided into three categories:

- *Coarse aggregate:* material retained by No. 4 sieve.
- *Fine aggregate:* material passing No. 4 sieve and retained on No. 200 sieve.
- *Microfines:* material passing No. 200 sieve.

Gradation plays an important role in the workability, segregation, and pump-ability of the concrete. Grading changes are more prevalent than shape and surface texture in the case of coarse aggregates. For example, uniformly distributed aggregates require less paste which will also decrease bleeding, creep and shrinkage while producing better workability, more durable concrete and higher packing [Quiroga and Fowler, 2004]. A graded aggregate, as opposed to a single-size aggregate, will have a greater packing density. The smaller aggregates will fill in the voids created by the larger aggregates [Graves, 2006]. Larger maximum sizes of coarse aggregates are beneficial for workability because they extend the range of aggregate sizes which improves grading [Fowler and Koehler, 2007]. Aggregate grading can be improved by combining two different grades of coarse aggregates. This practice is often used for pavement concrete in the Dallas and Fort Worth districts where a TxDOT grade 2 and a grade 4 [Table 3 of Item 421 of the TxDOT Manual] are combined to result in an improved grading. Improving aggregate grading can help maximize aggregate content and lower cement content.

Particles of irregular shape do not fit together perfectly and voids are created when these particles are assembled in a single container. The greater the void content, the more the paste required to fill these voids. The void content is affected by the particle size, shape, and grading. When a portion of two aggregates are combined and placed in a single container, the quantity of water (or paste) needed to fill the voids for the same volume decreases. Thus, combining aggregates of different size fractions reduce the void ratio.

Fine aggregate grading has a greater effect on workability of concrete than coarse aggregates [Quiroga and Fowler, 2004; Fowler et al., 2008]. Manufactured sands require more fines than natural sands to achieve the same level of workability; this is probably due to the angularity of the manufactured sands particles [Graves, 2006]. A decrease in the workability and durability of concrete are possible consequences of using an aggregate with either an excess or a lack of a particular size fraction [Galloway, 1994; Shilstone, 1990]. One common method used for evaluating gradation of fine aggregates is by computing the fineness modulus (ASTM C 33 or Tex-402-A). Fineness modulus is obtained by adding the total percentage of a fine aggregate sample retained on each of a specified series of sieves, and dividing the sum by 100. Various research studies have suggested that the fineness modulus is inadequate to differentiate between sands [Quiroga and Fowler, 2004].

Concrete mixtures with fine aggregate grading near the minimum for percent passing the No. 50 and No. 100 sieve may pose some problems with workability, pumping or excessive bleeding [ASTM C 33]. A fine aggregate that is too coarse will lead to harshness, bleeding, and segregation, but fine aggregate that is too fine will result in an increased water demand and segregation [Graves, 2006]. There is also an increase in water demand as dust of fracture (microfines) percentage is increased. This increase is attributed to an increase in the specific surface due to the particle size decrease [Ann and Fowler, 2001; O'Flynn, 2000]. The greater the maximum size aggregate in a mixture the less paste is needed, and the more the fine particles the more the paste required.

ASTM C 33 limits the microfine content to 7% for concrete, and 5% for concrete that is subject to abrasion. To meet ASTM C 33 requirements for aggregate passing the No. 200, the manufactured aggregate product that passes the No. 4 sieve (known as dry screenings) is conveyed to a wet sieving operation. The wet-sieved product is known as the manufactured sand. Research funded by ICAR has shown that good quality concrete can be produced using fine aggregate that does not meet ASTM C 33 standards [Fowler et al., 2008]. Compared to the same aggregate and grading without microfines, manufactured fine aggregates (MFA) with more than 17% microfines can be used to produce quality concrete that has the same or higher compressive and flexural strength, lower permeability, and higher resistance to abrasion [Fowler et al., 2008]. It should be noted that ASTM C 33 was developed for natural sands. The amount of microfines allowed by specifications has been limited for three reasons:

- Microfines may reduce workability due to large surface areas that need to be wetted. Microfines may increase the water requirement, which increases the amount of cement, therefore increasing shrinkage.
- Microfines tend to adhere to larger particles, preventing proper bonding between paste and aggregate. Improper bonding promotes cracking and weakens concrete.

- Clay particles may be present. These particles change volume when either they absorb or lose water. As a result, they expand when wet in fresh concrete and shrink when they dry in hardened concrete. Shrinkage increases cracking sensitivity, allowing for deleterious substances to ingress and reduce concrete strength [Katz and Baum, 2006].

Different limits than those required by ASTM C 33 can be found in specifications outside of the U.S. One example is the European Standard for Aggregates which allows up to 22% microfines content; however, should the content of microfines exceed 3%, the European specification requires testing for the presence of clay particles. On the other hand, the Israeli Standard for Concrete Aggregates limits the microfines content to 5% [Katz and Baum, 2006].

2.1.4 Absorption

Absorption is defined as the increase of mass due to presence of water in the pores of a material not including water adhering to the outside surface of a particle [ASTM C127; ASTM C128]. The absorption value may be regarded as an aggregate property that is a function of aggregate porosity and pore size [Yzenas, 2006]. It has been suggested that absorption might be a good indication of durability since it is a direct measure of accessible pore space in the aggregate [Forster, 2006]. However, this relationship has not been proven to be reliable [Forster, 2006]. Quiroga and Fowler found that the strength of the bond between cement and aggregate increases as absorption increases, but the durability decreases with an absorption increase [Quiroga and Fowler, 2004].

Some state transportation departments, such as the New Jersey Department of Transportation, specify a maximum absorption limit for aggregates. Such limits have been mainly been specified for coarse aggregates and not for fine aggregates. The problem with using a fine aggregate absorption value as a durability index is that the absorption value determined using ASTM C 128 (or using a similar test method) is not repeatable. The method for computing absorption by determining the saturated surface dry condition (SSD) for fine aggregates is very subjective. Rogers and Dziedziejko (2007) found that the presence of microfines results in greater multi-laboratory variation than obtained with the same group of laboratories when the fines are removed.

2.1.5 Mineralogy

The mineral composition of aggregates affects the performance of an aggregate in asphalt concrete as well as porltand cement concrete pavements. The main mineralogy performance issue related to pavements is skid resistance. The mineralogy of the aggregate also affects the shape and texture of crushed aggregates.

In asphalt concrete, it has been suggested that the presence of hard minerals is vital for producing polish resistant asphalt concrete [West et al., 2001; Masad et al., 2008]. Mohs hardness is a scale of mineral hardness that is based on the ability of one material to scratch another. The Mohs hardness values range from 1 to 10; a value of 1 represents a soft rock (Talc) and 10 represents the hardest known mineral (diamond). Carbonate aggregates have a Mohs hardness of 3, while rocks made of quartz have a Mohs hardness of 7. It should be noted that the hardness of carbonate/calcite aggregates can vary. Some carbonate aggregates such as dolomites have a higher Mohs hardness index value (around 3.5) [Alden, 2011]. Research on asphalt concrete has also shown that such aggregates (dolomites) have lower polishing susceptibility when compared to limestone aggregates [West et al., 2001].

The mineralogy of coarse aggregate is vital for obtaining good skid performance in asphalt concrete. In portland cement concrete however, the mineralogy of the fine aggregate is more important for obtaining good friction. NCHRP report 281 identifies fine aggregate mineralogy and hardness as important factors for obtaining good surface friction after the texture of a pavement is abraded [Folliard and Smith, 2003]. The coarse aggregate only becomes an influencing factor in cases where the top surface of the pavement has been severely abraded (or when coarse aggregate is intentionally exposed).

It is difficult to directly measure the resistance of fine aggregate to polishing [Folliard and Smith, 2003]. For this reason other indicator tests have been used to identify polish resistant fine aggregates. The most widely used test is the acid insoluble residue test (ASTM D 3042, in Texas Tex-612-J is used). The test assesses the presence of noncarbonated material in the fine aggregate; materials that have a high carbonate content yield a low residue because they dissolve in acid, while materials with low carbonate content yield a high residue. It is believed that the presence of acid insoluble material in the sand fraction generally improves skid resistance [Folliard and Smith, 2003]. In the 1950s Michigan banned the usage of carbonate fine aggregates in pavement concrete after very low friction numbers were measured on pavements made of the same source of fine and coarse limestone aggregate [Robords, 2008]. States such as Indiana and Minnesota have also banned carbonate fine aggregates in PCC Pavements; other states, including Texas, Illinois, Ohio, and Georgia have blended their carbonate fine aggregates with siliceous aggregates to avoid skid problems.

In general, the mineral composition of the majority of aggregates is naturally heterogeneous; it is therefore important to test for the presence of deleterious material that might have a negative impact on the performance of concrete. Deleterious materials might include clays, friable aggregates, chert, or organic materials [Forster, 2006]. In natural sands, it is important to determine the percentage of aggregates passing No. 200 sieve because those particles might be composed of deleterious materials such as clays [ASTM C 33]. Manufactured sands have a higher percentage of aggregates passing the No. 200 sieve that are not necessarily composed of clay particles [Fowler et al., 2008]. It is not enough to test the percentage of microfines present in a manufactured sand to identify the presence of clay; other tests such as the methylene blue (AASHTO TP 57) or the sand equivalent test (Tex-203-F) should be performed to test for the presence of clay particles in the microfines [Quiroga and Fowler, 2004; Fowler et al., 2008]. Another method of testing for the presence of clay in fine aggregates is the W.R. Grace methylene blue test. This test method uses a methylene blue solution to test the entire sample of fine aggregate using a colorimeter.

2.2 DURABILITY OF FINE AGGREGATES FOR PAVING CONCRETE

2.2.1 Acid Insoluble Residue Test

The main requirement for fine aggregates in paving concrete that is different from the requirements for all other uses of concrete is having a polish resistant aggregate. In Texas, the current limits are determined by the acid insoluble residue test (Tex-612-J). The TxDOT test consists of mixing a 25g sample of fine aggregate with a concentrated solution of hydraulic acid. After the reaction between the aggregate and the acid stops, the aggregate is washed and oven dried, and then the weight change is used to compute the acid insoluble residue. Individual aggregate sources have to meet an acid insoluble residue limit of 60%. If an aggregate does not meet this limit, then it has to be blended with another aggregate so that the blended fine aggregate meets the 60% acid insoluble residue limit. Prior to 1993, the minimum acid insoluble residue limit in Texas was 28%. This limit effectively omitted all carbonate fine aggregates. Between 1982 and 1993, some districts had started using higher requirements by plan note. The plan notes were not uniform, and the limits were based on sources local to each district. When the specifications were rewritten in 1993, the limit was set at 60% because that was representative of the value used by the districts [Herrera, 2011]. The only other state that has adopted the 60% acid insoluble residue limit is Oklahoma.

2.2.2 Magnesium Sulfate Test

The magnesium sulfate test is a test widely used to determine the durability of fine and coarse aggregates. In 1828 there was no method for freezing water in the laboratory, thus the sulfate soundness test was developed to simulate the forces generated by freezing water in building stone [Rogers et al., 1991]. The test is conducted with either sodium sulfate or magnesium sulfate. It consists of repeatedly re-immersing aggregates in a sulfate solution and then drying them; the mass loss is computed after the last drying cycle. The re-crystallization of salts inside the aggregate causes expansive forces inside the aggregate pores which simulate what happens during freezing and thawing when water freezes inside aggregate pores.

Most researchers agree that the test suffers poor repeatability, but conclusions on the ability of the sulfate test to predict field performance are mixed. Folliard and Smith (2003) recommend only using the magnesium sulfate test since it provides more precise values. In 1987, researchers determined that among seven laboratory test methods selected for the research the four-cycle soundness test was the best indicator of performance [Papaleontiou et al., 1987]. On the other hand, Kandhal and Parker (1998) claim that the crystal growth of salts inside the aggregate pores does not adequately simulate field conditions. Despite citing four references which show correlations with field performance, one researcher reported that sulfate soundness tests did not necessarily reflect field performance because stringent limits have been placed on a test that does not adequately model the actual field conditions of aggregate [Forster, 2006]. He explained that several sound aggregates have been rejected by sulfate soundness tests, and several unsound aggregates have been accepted by sulfate soundness tests and caused severe degradation in concrete. He concluded that sulfate soundness tests may be used to accept aggregate but should not necessarily be used to reject them. Some aggregates containing calcium or magnesium carbonate are attacked chemically by fresh sulfate solution, resulting in erroneously high measured losses [Meininger, 2002].

2.2.3 Micro-Deval

The micro-Deval test was developed to evaluate the wet mechanical strength and abrasion resistance of aggregates [Rogers et al., 1991]. The original test was invented in France, and its use in North America began in Canada where it was modified by the Ontario Ministry of Transportation (OMT). OMT developed tests for coarse and fine aggregates and because the micro-Deval test demonstrated good correlation with field performance, the Ontario Ministry of Transportation adopted the test for asphalt pavement, concrete, and granular base and sub-base applications [Rogers et al., 1991]. The test consists of placing a pre-soaked aggregate sample (washed and graded) in a jar with a fixed volume of water and a fixed quantity of steel ball bearings. The unit is then put into rotation for a specified period of time or number of cycles. After the sample is run in the device, it is washed over a sieve (No. 200 sieve for fine aggregates) and the retained sample is oven dried. The percent loss in mass is computed from the oven dried sample. Aggregates with a low percent loss are considered to be more durable than the aggregates with a higher percent loss. Aggregates that give more than 25% loss are considered to be marginal for use in portland cement concrete and asphaltic concrete

[Rogers, 1991]. ASTM D 7428 recommends a maximum micro-Deval percent loss limit of 20% for pavement concrete. The ASTM limit for structural concrete is also 20% loss.

Most of the research done on the fine aggregate micro-Deval test aimed to show that it can predict performance of fine aggregates better than the magnesium sulfate test. However in most cases the performance was not related to skid resistance of concrete pavement or even a quantifiable field or laboratory concrete performance criteria. Instead, experience-based evaluation of the general quality and performance of the material was compared to micro-Deval lab results. Rogers et al. (1991) found that the micro-Deval test for fine aggregates was more precise and repeatable than the magnesium sulfate test. Shabir et al. (2007) found that the micro-Deval fine aggregate test is better at predicting performance than the magnesium sulfate test. The performance criteria used by Shabir et al. was based on the experience of the Virginia Department of Transportation in using the selected aggregates for testing. Hudec and Boateng (1995) were able to relate micro-Deval percent loss to petrography. The micro-Deval loss values were influenced by the amount of shale or chert in the sample; higher contents of either led to higher micro-Deval percent loss.

2.3 PRODUCTION OF MANUFACTURED AGGREGATES

Most material for aggregate production comes from bedrock or unconsolidated deposits. These materials are obtained from surface-mined stone quarries or from sand and gravel pits. The mineralogy of an aggregate and extraction method affect the physical properties of the aggregate (shape, texture, and gradation); this in turn affects the physical and mechanical properties of the concrete produced with this aggregates. The major types of rocks used to produce crushed aggregates include limestone, granite, dolomite, trap rock, sandstone, quartz, and quartzite.

The production process begins by extracting the rocks either by drilling or blasting. The quarried rocks are transported to a processing plant and stored in large bins. To reduce the load on the primary crushers, screens are used to separate boulders from the finer rocks. Several types of crushers exist; the optimum choice of crusher is dependent on properties of the rock being crushed and on the reduction size required. The type of crusher also affects the shape of the crushed aggregate being produced. The following are the types of crushers commonly used in the production of aggregates:

- Jaw crushers
- Gyratory crushers
- Cone crushers
- Horizontal shaft impactors
- Vertical shaft impactors

Primary crushers are used for initial size reduction (6 to 12 in. in diameter); jaw, impactor, or gyratory crushers can be used as primary crushers. The rocks are then conveyed to scalping screens; rocks that are too large to pass the screens are processed through secondary and tertiary crushers (usually a cone or impact crusher). Secondary crushers reduce the size of the rocks to about 2 to 4 in.; tertiary crushers further reduce the size of the rocks to about 3/16 to 1 in. Oversized material from the tertiary crusher are sized in an inclined vibrating screen and processed in another cone crusher or a Hammermill (fines crusher) to further reduce the size of the rocks. The output of this operation is returned to the fines screen for resizing.

Compression crushers, like cone crushers, yield elongated shaped aggregates. However, this can be minimized by using a technique called "choke" feeding the crusher. Impact crushers, such as the Hammermill impactor tend to produce a uniform shape despite the higher operating cost. Centrifuge type crushing action in vertical impact crushers rounds sharp edges making manufactured sand particles similar to those of natural sands [Saunders, 1995]. One disadvantage of using impact crushers is that they produce more fines; these fines are usually washed to meet specifications (such as ASTM C 33). The process of washing aggregates to remove the fines increases the cost of the aggregates and leaves behind a large amount of unused materials.

The shape of the aggregate produced is affected by the speed on the crusher. The optimum speed for crushing an aggregate is highly dependent on the mineralogy of the aggregate. Prior to crushing rocks, aggregate producers run a series of tests to determine the crushing settings. Some of the properties that are determined prior to the crushing operation include abrasiveness and crushability.

2.4 BLENDED SANDS IN CONCRETE PAVEMENTS

When used in concrete pavements, soft manufactured sands such as carbonate sands polish and cause the concrete pavement to lose skid resistance. This is why manufactured sands are blended with siliceous sands. Many states have either used or adopted specifications for blended sands; these include Texas, Oklahoma, Illinois, Ohio, and Georgia. Some specifications require a minimum of 25% siliceous sand content in pavement concrete. In Texas, the combined blended sand has to meet an acid insoluble residue value of 60% (the test has to be performed on individual sands prior to blending). Since sands made of calcium carbonate are soluble in acid, the maximum percentage of manufactured carbonate sand cannot exceed 40%. Blending siliceous sands with manufactured sands has the following benefits:

- It has allowed softer manufactured sands to be used in concrete without negatively affecting skid performance.

- It has improved the grading of the fine aggregates, which in turn can improve the workability of the concrete.
- It has decreased the negative effect that poorly shaped manufactured sands have on the workability and finishability of concrete.
- It has reduced the cost of hauling large quantities of aggregates in locations where poor performing aggregates are present.

2.5 APPROACHES TO OPTIMIZING AGGREGATE GRADATION

2.5.1 Packing Density Methods

Packing density is defined as the volume of solids as a percentage of the total bulk volume. It provides an indirect mean of measuring aggregate geometric characteristics and a means of calculating the void content that needs to be filled with cement paste. Aggregate gradations with higher packing density allow for larger volumes of aggregates and lower volumes of paste.

Research done by Fuller and Thompson (1907) on adjusting gradation to render the greatest strength and workability concluded that aggregates should be graded in sizes and combined with water to give the greatest density. They developed a gradation curve that represented the greatest density of aggregates, but concluded that this gradation might not produce the greatest density when combined with cement and water because of the way cement particles fit in the pores. Work done by Wig et al. (1916) showed that the curve suggested by Fuller and Thompson (1907) does not always give the maximum density when aggregates different than the ones they studied were used. Talbot and Richart (1923) developed the following equation:

$$P = \left(\frac{d}{D}\right)^n \qquad \text{(eq. 2.1)}$$

where P is the amount of material in the system finer than size d, D is the maximum particle size, and n is the exponent governing the distribution of sizes. They concluded that for a given maximum particle size D, the maximum density can be achieved when n=0.5, but the resulting mixtures were harsh and not usable.

Many modifications have since been made to this equation; Shilstone (1990) and Quiroga (2003) suggested that the optimum value of n is 0.45. Work done by Bolomey (1947) extended the concept of parabolic grading and added an empirical value to the equation that reflected the desired level of workability. Furthermore, many other mathematical models based on empirical measurements have been developed to compute packing density.

2.5.2 Surface Area

According to Edwards (1918), the amount of water required for a concrete mixture is a function of the surface area of the aggregate particles. Young (1919) found that quantity of water required was dependent upon the quantity and consistency of the cement and the total surface area of the aggregate, which in turn is dependent on the grading. Young (1919) also found that the less the surface area of the aggregates, the less the excess water needed for the cement.

2.5.3 0.45 Power Chart

The 0.45 Power Chart is similar to a semi-log graph. It was originally used to obtain uniform gradation for asphalt mixture designs. The x-axis contains the sieve size, and the y-axis contains the percent of aggregates passing a given sieve. According to this method, the best combined grading, i.e. the grading with the least amount of voids is defined by a straight line. Fowler and Koehler (2007) used a modified 0.45 power chart for sands with high microfine content for optimizing self-consolidating concrete mixtures

(Figure 2.2). The difference between the modified 0.45 chart and the conventional 0.45 power is that the modified 0.45 power chart does not take into account microfines as part of the aggregate gradation (microfines are considered part of the paste portion).

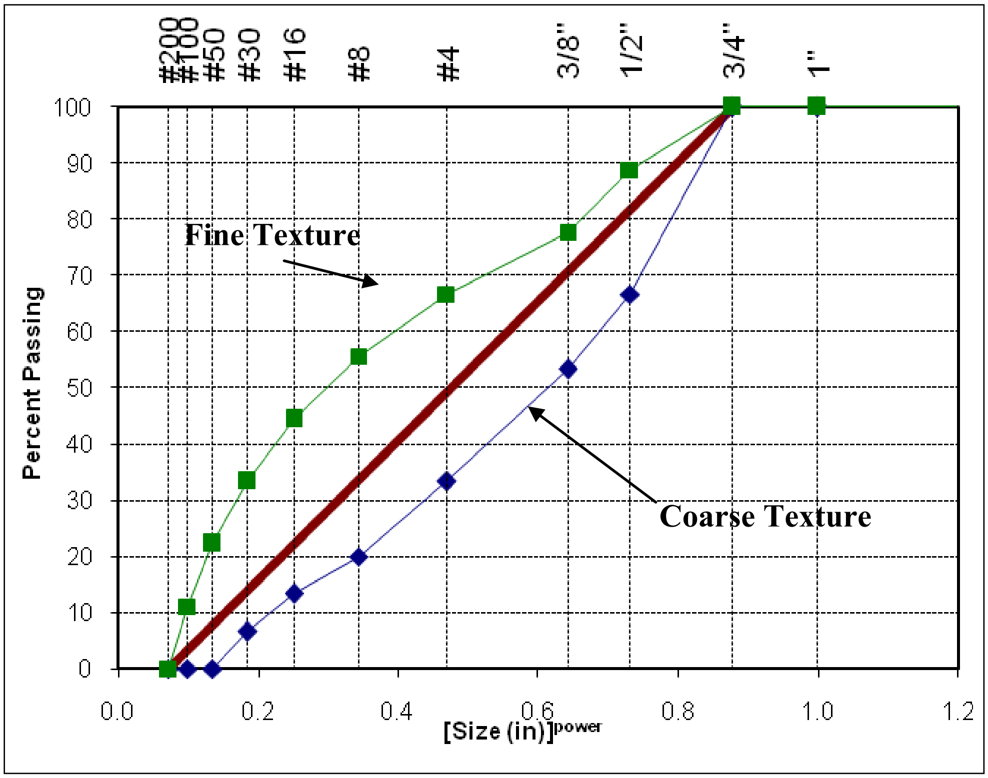

Figure 2.2: Modified 0.45 Power Chart [Koehler and Fowler 2007]

Deviations from the 0.45 power line help identify the location of grading problems. “Zigzags” across the line are undesirable. Gap-graded aggregate combinations will form an S-shape curve deviating from the optimum.

2.5.4 Coarseness Factor Chart

The coarseness factor chart developed by Shilstone (1990) is an alternative method of analyzing the size and uniformity of the combined aggregate particle distribution (Figure 2.3). For the coarseness factor chart a consideration of the grading of the whole aggregate is made, instead of considering the coarse and fine aggregate separately. Aggregate is divided into three fractions: large, Q, intermediate, I, and fine, W. Large aggregate is larger than 3/8-in., intermediate aggregate is considered to be between 3/8-in. and the No. 4 sieve and fine aggregate is defined as smaller than a No. 4 sieve and larger than a No. 200 sieve. All minus No. 200 sieve materials are classified as paste and the combination of paste and fine aggregate is considered mortar. The coarseness factor chart gives the relationship between the modified workability factor, which is equal to W corrected for cement content when more or less than 6 sacks per cubic yard are used, and the coarseness factor, which is defined as Q/(Q+I) [Quiroga and Fowler, 2004].

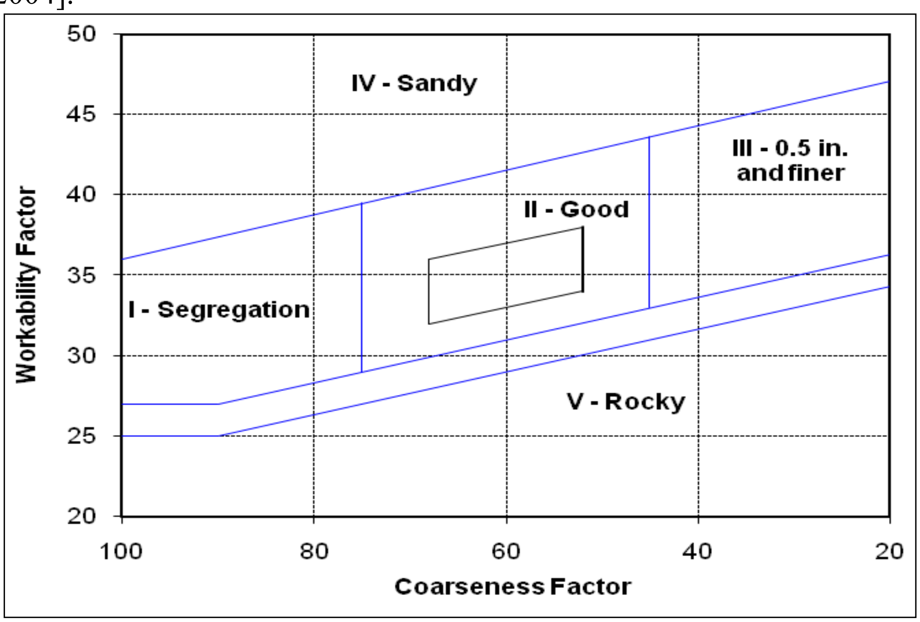

Figure 2.3: Coarseness Chart Proposed by Shilstone

This chart is based on the assumption that as cementitious materials are increased, the fine aggregate content should be reduced to maintain the same workability factor and vice versa. An increase or decrease in the cementitious materials or fine aggregate content without compensation in the other of these two components will impact the workability of the mixture. Five zones are defined in the chart:

- Zone I – This zone includes seriously gap-graded mixtures with high potential for segregation during placement or consolidation due to a deficiency in intermediate particles. They are not cohesive mixtures and are not recommended for paving or slabs due to segregation potential.
- Zone II – This is the optimum zone, including mixtures with nominal maximum aggregate size from 1-1/2 to 3/4 inch. These mixtures generally produce consistent, high quality concrete. Mixtures with slivered or flat intermediate aggregate require more fine sized aggregate due to their non-rounded shapes that create mobility problems.
- Zone III – This zone is an extension of Zone II for maximum aggregate size equal to or smaller than 1/2 inch.
- Zone IV – These mixtures have excessive fines leading to a high potential for segregation during consolidation and finishing. Mixtures in this zone will produce variable strength, have high permeability and exhibit shrinkage.
- Zone V – Mixtures falling in this zone are very coarse or non-plastic, creating a need to increase the fines content (ACI 302-04; IM 532).

2.5.5 Percent Retained

The current ASTM C 33 specification could lead to poor workability mixtures and gap-graded mixtures due to an excess or a deficiency of some sizes. The goal of the

"Percent Retained" method (Figure 2.4), sometimes referred to as the "18-8" method, is to produce uniform blends by limiting the maximum and minimum amount of aggregate fractions to a ceiling value of 18% and a floor value of 8% [Quiroga and Fowler, 2004].

A deficit in particles retained on the No. 8, 16 and 30 sieves and an excess of particles retained in the No. 50 and 100 sieves can be found in many areas of the U.S. This leads to problems such as cracking, curling, blistering and spalling of concrete. If there is a deficit in one sieve but an excess on the adjacent sieve, the two sieve sizes can balance one another. However, if there are three adjacent deficient sieve sizes, the grading distribution in these sieves needs to be adjusted. These deficits can be seen through adjacent peaks and dips in the "18-8" chart [ACI 302-04; IM 532].

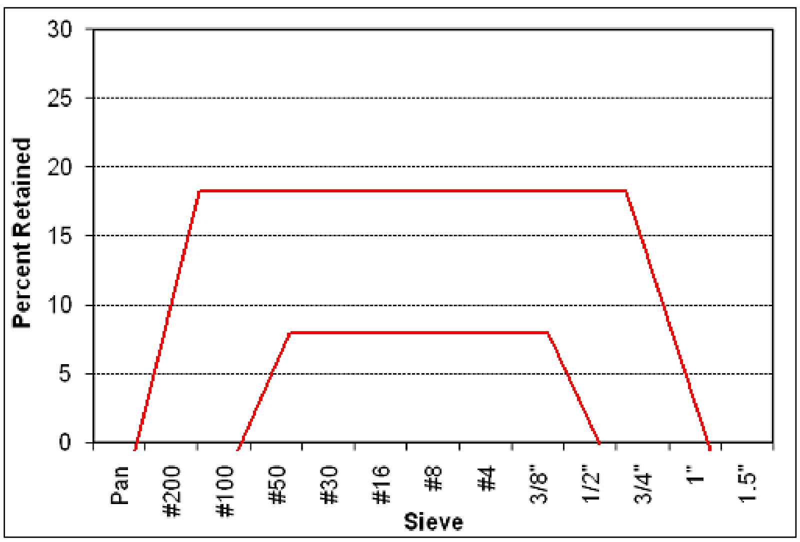

Figure 2.4: "18-8" Percent Retained Chart

This method however is not intended to be used for aggregate with high microfines content. The mixtures meeting the "18-8" limits could still have workability problems and low packing density due to an excess or deficit of either fine or coarse aggregate [Quiroga and Fowler, 2004].

Chapter 3: Concrete Properties and Performance Literature Review

The main acceptance criterion for aggregates should be related to their performance in concrete. If good quality concrete that meets all the required performance criteria can be made using a certain source of aggregate, then there are no reasons to not allow that aggregate to be used in concrete.

This chapter will discuss how aggregate properties affect fresh and hardened concrete performance. Methods of evaluating skid resistance and proportioning concrete are also reviewed in this chapter.

3.1 EFFECT OF FINE AGGREGATES ON FRESH CONCRETE PROPERTIES

Particle shape, texture, and grading have a great impact on the fresh properties of concrete. Mixtures containing high amounts of poorly shaped particles (like MFAs) tend to need a higher amount of paste content to achieve the same workability (compared to a mixture, made with natural sands) [Fowler et al., 2008]. Other properties such as finishability, air content, bleeding, and segregation might also be affected by the use of MFA.

3.1.1 Workability

In the 1970s, an aggregate manufacturer in North Carolina began promoting the use of manufactured sands in pavement concrete. A test section was made with manufactured sand containing a maximum of 3% microfines. The performance of this manufactured sand was a nightmare for the paving contractor [Saunders, 1995]. The concrete workability was horrible; there was excessive bleeding, edge slump, and edge shearing [Saunders, 1995]. Following this incident, a 50/50 blend of manufactured sand and natural sand was used instead to improve performance.

Fine aggregates have a higher impact on workability than coarse aggregates (Wills, 1972). One of the obstacles to using MFA in concrete is that manufactured sands are typically composed of sharp, angular particles with large numbers of flat and elongated particles [Graves, 2006]. Angular particles create a greater void volume within the aggregate. Additional paste (water and cement) is needed to fill those voids [Quiroga and Fowler, 2004]. This, however, can be offset by using a higher dosage of admixture [Fowler et al., 2008]. When using MFA in concrete mixtures, a water-reducing admixture may not be sufficient to achieve a slump of 2 in. [Trachet, 2008]. Mid-range or high-range water-reducing admixtures (MRWRA & HRWRA) have a higher water reducing capacity; however MRWRA and HRWRA are not usually used for slipform paving jobs.

Another aspect of MFA that affects workability is the presence of high amounts of microfines. Microfines are believed to have an adverse effect on the workability of concrete due to their small sizes (large surface area) and because they might contain deleterious materials (like clay and other organic materials). Research by ICAR on self-consolidating concrete found that microfines can be successfully used and can lead to an improvement in the workability of concrete (when low amounts of deleterious materials such as clays are present) [Koehler and Fowler, 2007]. Furthermore, when microfines are considered as part of the aggregates, higher dosages of admixture are needed to achieve the same workability as compared to mixtures where the microfines are accounted for as part of the paste [Fowler et al., 2008].

Both the angular nature of MFA and the presence of high amount of microfines affect the workability of concrete. These negative impacts on workability can be counteracted by blending sands, using an admixture, or by the addition of fly ash [Trachet 2008]. Increasing the quantity of manufactured sand in a blend will reduce workability or will require a higher dosage of admixture [Trachet, 2008].

3.1.2 Finishability

One of the problems experienced during the "TxDOT Implementation Project for Increased Microfines Content in Pavement Concrete" was problems that involved finishing [McLeroy, 2008]. The finishing crew tried to solve the problem by adding more water to the surface of the concrete ("blessing" the concrete). Such problems have not been encountered when natural sands were used, and are believed to have been caused due to the poor shape of MFA and the presence of high amounts of microfines [McLeroy, 2008]. A type D admixture (water reducing and retarding) was used in that project. Finishability can be improved by either improving the shape or grading of crushed aggregates [Saunders, 1995] or by using a different type of admixture. MRWRA admixtures have higher water reducing capacity, and also have the ability to improve surface slickness, which results in easier finishing and better concrete surfaces [Schaefer, 1995]. Using a MRWRA might help solve workability and finishability problems encountered when MFA are used, but such admixtures are not usually used in paving concrete.

3.1.3 Bleeding and Segregation

By increasing the water demand, the amount of bleed water in the concrete increases [Washa, 1998; Kosmatka, 1994]. Research done during ICAR 401 [Fowler et al., 2008], have shown that for the same mixture proportions, mixtures containing MFA will only bleed if not enough paste (water and cement) is present in the mixture; the volume of paste to avoid bleeding in mixtures containing natural sands is lower than that of mixtures made with MFA. As for segregation, Kalcheff (1977), Hudson (1999), and research done by the Japan Society of Civil Engineers (2002), have shown that the presence of microfines help decrease the segregation of concrete.

3.1.4 Air Content

Research done during ICAR 104 suggest that the high percentage of microfines present in MFA can lead to an increase in the amount of entrained air, and thus decrease the amount of air-entrainment needed [Quiroga and Fowler, 2004]. Later research done on MFA has shown no correlations between the presence of MFA and the increase of entrained air [Trachet, 2008]. The observed increase in entrained air might be due to the increase in the dosage of the water-reducer, or maybe due to the presence of fly ash [Trachet, 2008].

3.2 THE EFFECT OF FINE AGGREGATES ON HARDENED CONCRETE PROPERTIES

The hardened properties of concrete are affected by the amount, mineralogy, and grading of aggregates. The addition of water to mixtures made with MFA to compensate for the water needed to achieve the required workability might affect the strength and durability. Properties that might be affected by the use of MFA include compressive and tensile/flexural strength, shrinkage, permeability, and skid resistance.

3.2.1 Strength

Using water-reducing admixtures, concrete mixtures made with MFA do not need additional water to achieve the required workability. ICAR 401 [Fowler el al. 2008] showed that the same compressive strength can be produced by using natural sand, a well-shaped manufactured sand, or a poorly-shaped manufactured sand. The only additional requirement was a higher dosage of HRWRA that was used to achieve the same workability. Research by Kim et al. (1997) has shown that concrete made with crushed limestone fines has generated compressive and tensile strengths about equal to or larger than natural sand. Other research by Celik and Marar (1996) has shown that the strength is affected by the percent of microfines in the sand; fine aggregates with amounts of microfines equal of about 30% lead to a decrease in compressive and flexural strength. For the same water-to-cement ratio, manufactured fine aggregates or blended sands demonstrate either equal or superior strength or resistance to carbonation, when compared to concrete made with natural sand [Yamamoto et al., 2005]. Using manufactured granite sand in a blend can cause reduction in compressive and flexural strength [Trachet, 2008], while using dolomitic limestone sand increases strength [Trachet, 2008]. The granite sand tested by Trachet (2008) produced concrete with the lowest values of compressive strength and flexural strength.

3.2.2 Shrinkage

Due to the restraining effect of aggregate particles, concrete generally shrinks less than cement paste. According to Torben el al. (1965), the degree of restraint provided by the aggregates in concrete is dependent on the quantity of aggregate, elastic properties of the aggregate, and the shrinkage of the cement paste and aggregate; the greater the volume of the aggregate in the concrete, the less the shrinkage. Furthermore, the lower the aggregate modulus of elasticity the lower the restraining effect on the cement paste during shrinkage. Higher shrinkage is usually associated with higher paste content (water and cement) [Fowler et al., 2008]. When microfines in manufactured sands are considered as part of the paste, rather than as part of the aggregates, shrinkage decreases; this is due to decrease of cement in the mixtures [Fowler et al., 2008]. Higher shrinkage in mixtures containing manufactured sands can be attributed to the higher paste requirements.

3.2.3 Skid Resistance

Skidding, slipping, or sliding occurs as a result of lack or loss of friction. Friction is defined as the force resisting relative motion of two objects in contact (Figure 3.1).

Friction created by the tire and pavement interaction allows vehicles to accelerate, decelerate, and maneuver. A study done by Viner et al (2004) estimated that risks of accidents crashes can be halved by doubling the skid resistance [Hall et al., 2006]

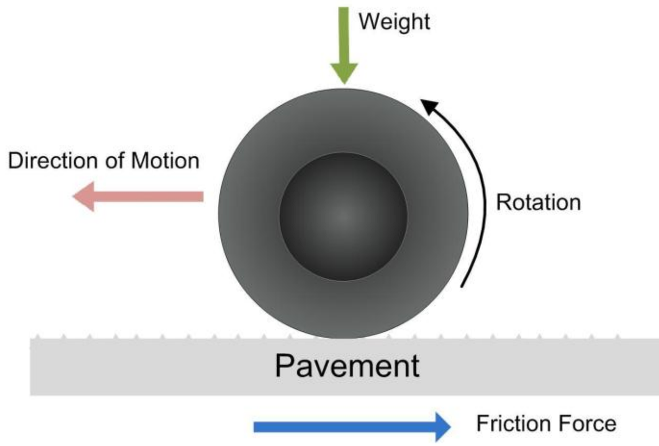

Figure 3.1: Friction Force

In PCC pavements, adequate surface friction generally exists in dry conditions. In wet conditions the presence of water reduces the contact between the tire and the pavement, which reduces friction. If a sufficiently thick film of water is formed between the tire and the pavement (this might occur at higher traveling speeds), the tire will lose contact with the pavement, a phenomenon known as hydroplaning (Figure 3.2) [Hoerner et al., 2003]. The difference between hydroplaning and skidding is that in hydroplaning there is no contact between the tire and the road surface to develop any frictional force.

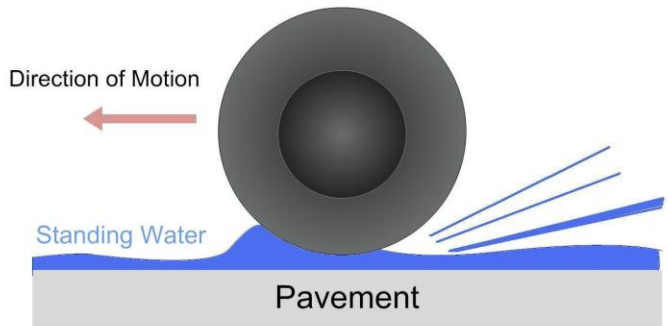

Figure 3.2: Hydroplaning

Early investigations of friction hypothesized that friction was only due to the interlocking of mechanical protuberances or asperities on the surfaces of contacting materials [Rabinowicz, 1995]. This explanation for the friction phenomena is referred two as the “roughness hypothesis” [Rabinowicz, 1995]. In the 1900s, as the science of surface chemistry was developed, it became evident that friction was not just due to roughness.

The friction force between two objects arises from the need to shear strongly adherent surface atoms of contacting materials [Rabinowicz, 1995]. This phenomenon is known as adhesion and usually accounts for 90% or more of the overall friction force [Rabinowicz, 1995]. On wet pavements, adhesion can account for two-thirds of the resistance force [Hogervorst, 1974]. The other main factor contributing to the frictional force involves the roughness of a surface [Rabinowicz, 1995]. This component of friction arises from the need during the sliding of rough surfaces to lift one surface over the roughness of the other. This phenomenon is also known as the hysteresis component. A depiction of adhesion and hysteresis between tire and pavement is shown in Figure 3.3.

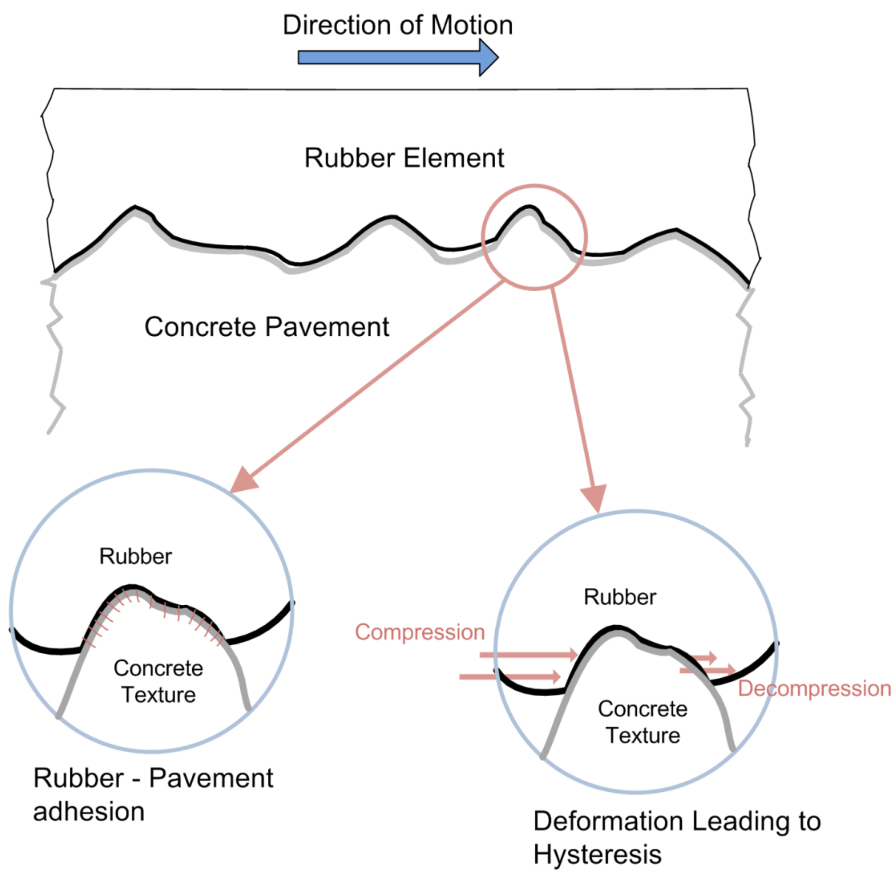

Figure 3.3: Schematic Plot of Adhesion and Hysteresis [adapted from Hall et al., 2006]

The adhesion component of the friction force is proportional to the area of contact [Rabinowicz, 1995]. The area of contact though is not the "apparent area" (or the visible area) but the "real area" of contact. The real area of contact might be larger than the apparent area of contact because it is made up of large number of small regions of contact, or "junctions" (not necessary visible). To prove that friction is not necessarily related to surface roughness but is rather related to adhesion, Bailey and Courtney-Pratt (1955) showed that atomically smooth surfaces of mica, produced by cleavage, show very high friction [Rabinowicz, 1995].

3.2.3.1 Textures for Concrete Pavements

Texture on pavements is composed of the deviations of the pavement surface from a true planar surface. Many types of texturing methods are used in concrete, some are formed in wet concrete and others are formed in hardened concrete. Textures formed in wet concrete include texture formed by dragging techniques (burlap, carpet, broom, etc…), tining (longitudinal or transverse), or exposed coarse aggregate (less commonly used). Textures formed in hardened concrete include ground concrete (diamond grinding) or shot blasted/abraded concrete (less commonly used).

The Permanent International Association of Road Congresses (PIARC) defines three levels of texture; these are micro-texture, macro-texture, and mega-texture [Hall et al., 2009]. Each of these can be differentiated by their wavelength [λ] and amplitude (A)[Hall et al., 2009].

Micro-texture (λ < 0.5 mm, A = 1 to 500 μm), is the surface roughness on the microscopic level. Unless the coarse aggregate is exposed, the micro-texture is mainly influenced by the fine aggregate in PCC pavements. The micro-texture is important to maintain adequate friction in dry-weather conditions and wet-weather conditions when speeds are under 72 km/h (45 mph) [Hall et al., 2009].

Macro-texture (0.5 mm ≤ λ < 50 mm, A = 0.1 to 20 mm) is defined by the type of surface finishing/texturing technique formed in the surface of the concrete. Good macro-texture is required to maintain adequate friction under wet-weather conditions at speeds over 72 km/h (45 mph) and to prevent hydroplaning.

Mega-texture (50 mm ≤ λ < 500 mm, A = 0.1 to 50 mm). This type of texture is usually undesirable and is unintentionally formed or is a result for distress in concrete.

Texture can also influence properties of the pavement other than friction. These properties include noise, ride quality, splash and spray and tire wear. A summary of how the different levels of texture can affect concrete pavement is presented in Figure 3.4. In Figure 3.5, aggregate texture, fine aggregate size, and coarse aggregate size are compared to the wavelength of the different levels of texture on a pavement.

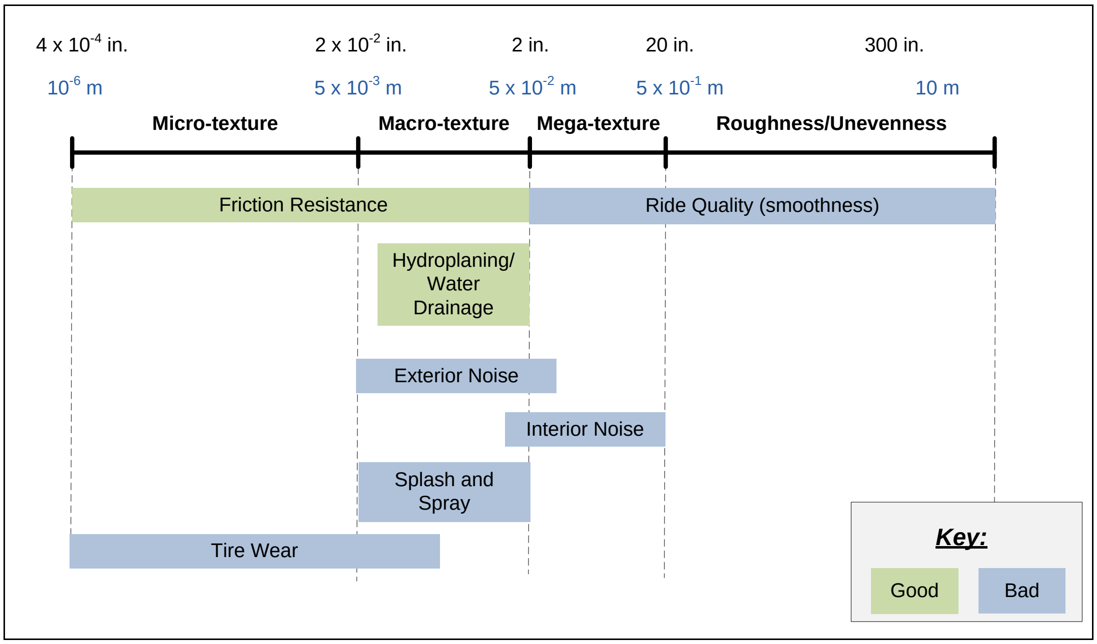

Figure 3.4: Pavement Wavelength and Surface Characteristics [adapted from Hall et al., 2006; Hoerner, 2003]

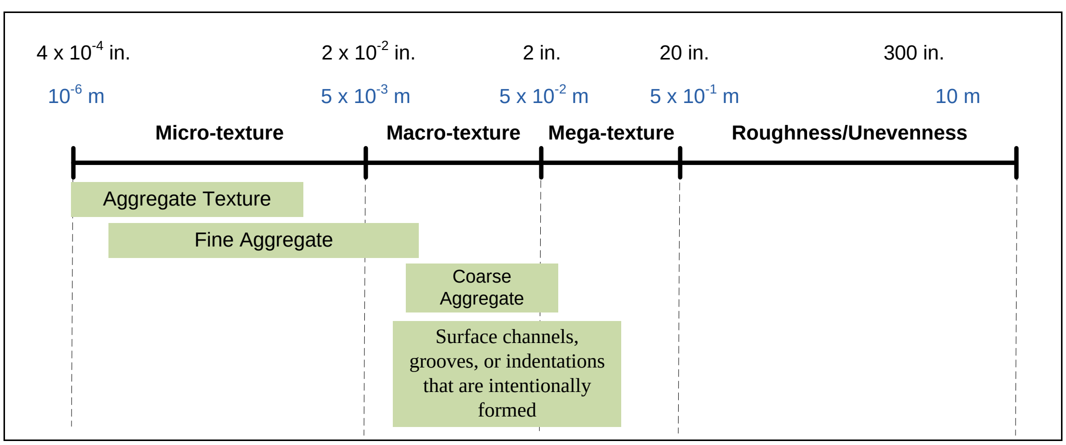

Figure 3.5: Type of Texture Contributing to Texture

3.2.3.2 Factors Affecting the Skid Resistance of Portland Cement Concrete Pavement

After the texture formed on the concrete surface is abraded (mainly the texture created by dragging techniques), the skid resistance of a pavement is a function of the fine aggregate used in the concrete mixture. This property of PCC pavement was recognized early on and research has been done to find tests that better evaluate the performance of fine aggregates for skid.

According to Balmer and Colley (1966), the need for skid resistant pavements was recognized in 1958 by the First International Skid Prevention Conference. After this conference, state agencies started to develop equipment to test skid both in the laboratory and in the field. In 1958, Shupe and Lounsbury showed a correlation between calcium carbonate content of aggregates and skidding susceptibility. Gray and Renninger (1965) recognized the contribution of siliceous sand particles in skid resistance and pioneered the acid insoluble residue test to analyze the amount of siliceous materials in the aggregates. Balmer and Colley (1966) compared acid insoluble residue of fine aggregates to a laboratory skid performance test. It should be noted, however, that the acid insoluble test used by Balmer and Colley differed significantly from what is currently being used by TxDOT. Balmer and Colley tested samples that had a weight ranging from 1 to 2 lbs. (450 to 900 grams) with a 6N solution of hydrochloric acid solution. The TxDOT test method uses 25 grams of fine aggregates along with a concentrated hydrochloric acid solution. ASTM D 3042 (Standard Test Method for Insoluble Residue in Carbonate Aggregates) is similar to the test conducted by Balmer and Colley.

The laboratory testing conducted by Balmer and Colley consisted of subjecting concrete specimen made with sands having different mineralogy to three cycles of wear. The first and third cycles consisted of wearing the surface by means of a rotating 600 lb loaded tire. The second cycle was similar to the first and third cycles, but was complemented by the addition of fine Ottawa sand. The goal of adding the fine aggregate was to simulate wear caused by the grit and dirt on roadways. The results obtained from Balmer and Colley are shown in Table 3.1.

Fine Aggregate No.	Principal Constituents, %	Rating of Field Performance	Wear Index, kw
1......	90 calcite	poor	3.6
2......	70 calcite 24 dolomite	poor	4.4
3......	90 calcite	poor	4.0
4......	80 dolomite	poor	5.6
5......	75 dolomite	poor	5.8
6......	70 dolomite	poor	5.4
7......	60 calcite 16 silt and clay	poor	5.3
8......	80 calcite 15 quartz	fair	6.2
9......	65 calcite 12 dolomite	poor	5.9
10......	50 calcite 33 mica and quartz	excellent	6.8
11......	55 dolomite 39 quartz, quartzite, and feldspar	excellent	6.8
12......	55 calcite and dolomite 40 quartz, mica, and epidote	excellent	6.7
13......	45 calcite 42 quartz and feldspar	excellent	7.3
14......	50 dolomite 44 quartz	excellent	7.0
15......	45 dolomite 45 quartz	excellent	6.8
16......	45 dolomite 45 quartz	excellent	7.2
17......	30 graywacke 55 quartz	excellent	7.0
18......	75 quartz 17 feldspar	excellent	7.3
19......	72 quartz 20 feldspar	excellent	7.2
20......	99 quartz	excellent	7.5

Table 3.1: Wear Index Results Obtained by Balmer and Colley (1966)

Balmer and Colley rated the skid performance of the concrete by measuring the power required to rotate a wheel against the abraded specimen. This value was translated into a wear index. The wear index values ranged from 3.5 to 7.5 (Table 3.1); the "calcites" (probably limestone aggregates) had the lowest values, while the aggregates containing quartz (siliceous) had the highest. Although dolomitic aggregates were rated as "poor", they had values ranging from 4.4 to 5.8.

After comparing the results obtained from the aggregate tests to the concrete laboratory performance test, Balmer and Colley concluded that 25% siliceous fine aggregate replacement was satisfactory for performance with most aggregates (Figure 3.6).

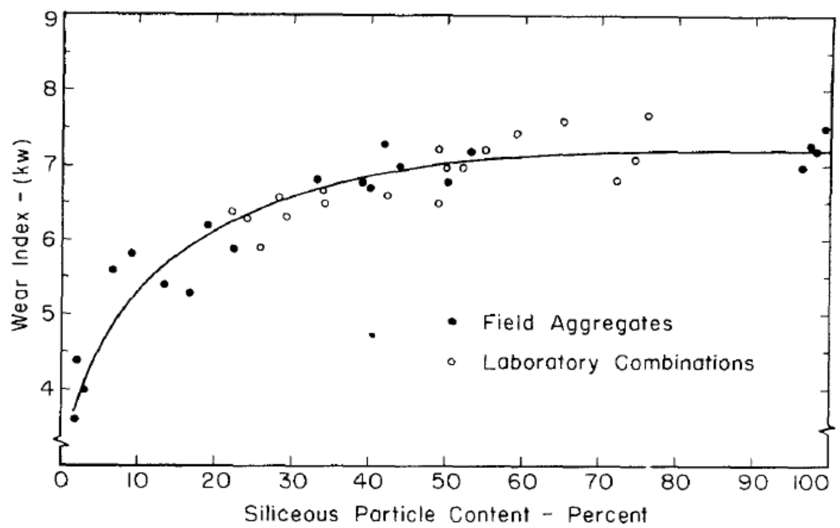

Figure 3.6: Effect of Siliceous Particle Content on Wear Index [Balmer and Colley, 1966]

Most current specifications base their limits on the study done by Balmer and Colley (1966). Federal Highway Administration guidelines recommend the usage of wear resistant aggregate. FHWA recommends a minimum siliceous fine aggregate content of 25% [FHWA, 2005]. Such a limit would allow up to 75% of the sand to be composed of carbonate aggregate (35% more than the TxDOT specifications).

Most of the studies done after 1966 had the same conclusions as the study done Balmer and Colley. Renninger and Nichols (1977) found good correlation between skid resistance (as determined by the British Pendulum Tester) and acid insoluble residue.
As part of a study the evaluated micro-texture and macro-texture on PCC pavements around the United States, Hall and Smith (2009) found that tougher, more durable aggregates retain higher friction values. They found that the usage of limestone in Kansas and Illinois resulted in greater rates of micro-texture deterioration compared to the usage of high silica granite in Minnesota.

In Pennsylvania, PDOT formed a committee to investigate decrease in skid resistance on some PCC pavements that was attributed to the use of a soft limestone coarse aggregate. A task force was formed in 2006, after several crash clusters in areas of I-80 were reported. The committee determined that the pavements with diminished skid performance had lost the surface mortar and that the tires were riding in a combination of coarse aggregate and mortar. The “loss of surface” (loss of the mortar on the surface) was attributed to the usage of metal stud/chain, diamond grinding, or shot blasting of the surface [PDOT, 2007].

The properties that affect PCC pavements and asphalt concrete skid performance are similar but are due to different factors. Table 3.2 is a summary of how PCC and asphalt pavements differ.

Property/Factor	**PCC Pavement**	**Asphalt Concrete Pavement**
Aggregates contributing to skid resistance	Unless the coarse aggregate is exposed, skid resistance is mainly affected by the fine aggregate.	The coarse aggregate plays a major role in skid resistance; fine aggregate have little to no effect.

Aggregate property affecting skid resistance	Fine aggregate mineralogy is the main factor in determining the long-term skid resistance of PCC pavements.	Coarse aggregate mineralogy, shape, angularity, and texture affect performance.
Macro-texture	Macro-texture is formed on the concrete surface to drain the pavement from water and to avoid hydroplaning; it is formed mainly by tining or grooving.	Macro-texture is not intentionally formed; it is defined by the angularity and shape of the coarse aggregate.
Loss of skid (deterioration)	Wear or loss of friction is mainly due to abrasion.	Other factors might affect the loss of skid resistance besides abrasion (such as temperature, age, etc…).

Table 3.2: Properties and Factors Affecting the Skid Resistance of PCC and Asphalt Pavements

3.3 EVALUATING PAVEMENT SKID PERFORMANCE

Many methods have been developed to evaluate skid resistance both for field and laboratory usage. These methods either evaluate micro-texture, macro-texture, or both. In this section, methods of evaluating friction, texture, and equipment used to simulate wear at a laboratory will be discussed.

3.3.1 Test Methods for Evaluating Texture

The Sand Patch Method (ASTM E 965 or Tex-436-A) is a method used to measure the average macro-texture depth of a pavement. The test consists of spreading a uniform material of known volume on a clean and dry pavement surface and then calculating the average depth of the macro-texture based on the area covered by the material. The sand test method is known to be cumbersome and has poor repeatability [Doty, 1974]. Another method similar to the sand patch method was developed, it is known as the grease patch method.

The Outflow Meter (ASTM E 2380) is also a method used to evaluate macro-texture. It consists of measuring the time it takes for a cylinder of known volume to discharge water over a pavement. This method is suitable as a field test to evaluate surface drainage.

The Circular Track Meter (CTM - ASTM E 2157) is a device that utilizes a displacement sensor that is mounted on an arm that rotates in a circular path and measures the mean profile depth (MPD) of a pavement (macro-texture). The CTM is a device that can be used in the field and in a laboratory to evaluate macro-texture.

Many other methods that use laser or image processing equipment like the CTM have been developed. Some of these include the Road Surface Analyzer (ROSAN), Robotex, the Multi-Laser Profiler (MLP), the Lightweight Profiler, and the Liteweight Inertial Surface Analyzer (LISA). Many of those devices were developed because highway agencies were researching methods of evaluating pavement texture and skid resistance without interrupting traffic.

3.3.2 Test Methods for Evaluating Friction

The Locked-Wheel Skid Trailer (ASTM E 274) is the most common method used by state agencies to evaluated skid resistance. The method consists of measuring the locked-wheel friction (100% slip condition) of a trailer towed behind a truck at a speed of 40 mph (TxDOT uses 50 mph). The trailer administers a water spray to the pavement in front of the tire to simulate wet conditions. The resulting friction force acting between the test tire and the pavement surface is used to determine the skid resistance which is reported as a skid number (SN). Higher SN values signify higher skid resistance. A smooth tire (ASTM E 524) or a ribbed tire (ASTM E 501) can be used on the skid trailer. Research has shown that ribbed tires are only capable of evaluating the effect of micro-texture on friction, while smooth tires can measure the contribution of micro-texture as well as macro-texture [Jackson, 2008; Hall et al., 2006]. Some state agencies have trigger skid values that they use as means of initiating some sort of rehabilitation treatment; these values differ from state to state. The most common trigger values reported are SN < 35 or 30 for ribbed tires, and SN < 20 for smooth tires [Hall et al., 2006]. It is believed that SN values below those limits can result in an increase in skid related accidents on roadways.

The British Pendulum Tester (BPT - ASTM E 303) is one of the simplest and cheapest instruments used in the measurement of friction characteristics of pavement surfaces [Lee, 2005].The BPT produces a low-speed sliding contact (about 10 km/hr) between a standard rubber slider and a pavement surface. The elevation to which the arm swings after contact is used to compute a frictional value. Various studies have shown that the BPT is unreliable especially when used on coarse-textured surfaces [Lee, 2005].

The Dynamic Friction Tester (DFT - ASTM E 1911) is laboratory and field apparatus that measures the friction-speed relationship on a pavement surface for speeds ranging from 0 to 80 km/hr. The DFT measures the torque needed to stop three small spring-loaded standard rubber pads rotating in a circular path. The torque measured is then converted to a friction value. Water is also introduced during testing to simulate wet conditions. The DFT can be used along with the CTM to evaluate both micro-texture and macro-texture on the same circular path.

Since the 1960s many methods of measuring frictional resistance have been invented worldwide. Most of these methods were mainly developed for field usage and they closely resemble the Locked-Wheel Skid Trailer discussed earlier; these include the British Mu-Meter, the British Sideway Force Coefficient Routine Investigation Machine (SCRIM), Roadway and runway friction testers (RFTs), the Airport Surface Friction Tester (ASFT), the Saab Friction Tester (SFT), the Griptester, the Finland BV-11, the Road Analyzer and Recorder (ROAR), and the Norwegian Norsemeter RUNAR [Hall et al., 2006]. Other methods of evaluating friction are based on measuring the stopping distance (ASTM E 445) or the deceleration rate (ASTM E 2101). Some of these methods are used to evaluate friction at different speeds while an antilock braking system (ABS) is fully engaged. Laboratory sized equipment similar to the BPT and DFT have also been developed; these include the Michigan Laboratory Friction Tester, the North Carolina Variable Speed Friction Tester, and the Pennsylvania Transportation Institute (PTI) Tester.

3.3.3 Accelerated Wear and Polishing Devices

Machines that simulate wear and polish caused by traffic have been used since research on skid resistance started in the 1960s. Some were made to wear and polish aggregates, while others were made to wear and polish asphalt and concrete surfaces. Such devices include:

- The British Polishing Wheel.
- The Michigan Indoor Wear Track.
- The micro-Deval.
- The Three-Wheel Polishing Device (TWPD) developed by the National Center for Asphalt Technology (NCAT).
- The North Carolina State University Wear and Polishing Machine.
- The Wehner/Schulze Polishing Machine.

- The Penn State Reciprocating Polishing Machine.
- The Model Mobile Load Simulator (MMLS-3).

All those machines essentially do the same thing; they wear and polish aggregate, asphalt, or concrete surfaces. In general these devices differ in the following:

- The size of the machine and the area it polishes.
- The material used to polish specimen (Different types of tire material have been used to abrade the surfaces).
- Some devices have used means to accelerate wearing and polishing. For example, hard sands have been used to complement and accelerate the wear caused by tires.
- Some devices utilize water in the abrasion process, while others do not.

It should be noted that most of those devices have been developed for testing asphalt pavements and not for concrete pavements. Moreover, some of those devices have shown to reproduce wearing and polish patterns very similar to what is observed in the field; such machines can be used to estimate when the loss of skid will occur based on the materials used, expected traffic, and age of a pavement.

3.3.4 Correlating Skid Values Measured by Different Devices

Many methods and devices of evaluating skid resistance have been created, and each of these evaluates skid in its own defined way. The need to define a common scale for friction on pavements was explored by the Permanent International Association of Road Congresses (PIARC). In 1992, PIARC conducted a study aimed at correlating the results obtained from 51 different measurement systems used worldwide. Sixteen countries covering each continent participated and experiments were conducting at 54 sites across the U.S and Europe. One of the main results of the PIARC experiment was the development of the International Friction Index (IFI) [PIARC, 1995]. ASTM E 1960 defines the IFI as an index for comparing and harmonizing friction measurements with different equipment to a common calibrated index. For example values measured using a CTM and DFT can be used to calculate the IFI index; the IFI index can then be used to compute the equivalent texture or friction values for other devices. The following is an example of how values measured using a CTM and DFT can be converted to equivalent locked-wheel skid trailer skid numbers (SN – ASTM E 274):

- The IFI parameters F60 and S_p are first computed using the formulas provided in ASTM 1960:

$$S_P = 14.2 + 89.7MPD \qquad \text{(eq. 3.1)}$$

$$F60 = A + B \times FRS \times e^{(S-60)/S_p} \qquad \text{(eq. 3.2)}$$

- A and B are the calibrated constants for the device used for measurement. A = 0.0082 and B = 0.732 for the coefficient of friction measured by the DFT at 20 km/hr (ASTM 1960). FRS is the friction value measured at a speed S (in this case at 20 km/hr). The MPD is the mean profile depth value measured by the CTM.

$$F60 = 0.082 + 0.732DFT_{20}e^{-40/S_p} \qquad \text{(eq. 3.2)}$$

- After the IFI parameters F60 and S_p are computed, the same formulas using different device constants can be used to compute the equivalent friction.
- To compute the equivalent skid number (SN) measured by a locked-wheel skid trailer at 40 mph using a smooth tire (ASTM E 524), the following formula can be used:

$$SN(40)_{Smooth} = \left(\frac{F60-0.045}{0.925} \times \frac{1}{e^{\frac{4.37}{S_P}}}\right) \times 100 \qquad \text{(eq. 3.3)}$$

- To compute the equivalent skid number (SN) measured by a locked-wheel skid trailer at 40 mph using a ribbed tire (ASTM E 501), the following formula can be used:

$$SN(40)_{ribbed} = \left(\frac{F60+0.023-0.098\times MPD}{0.607} \times \frac{1}{e^{\frac{4.37}{S_P}}}\right) \times 100 \qquad \text{(eq. 3.4)}$$

Research done at University of North Florida on asphalt concrete found direct correlation between the DFT coefficient of friction at 60 km/hr and the locked-wheel skid trailer value measured at 40 mph using ribbed tires [Jackson, 2008]. A similar correlation for asphalt concrete was found by NCAT after correlating field and laboratory results (Figure 3.7) [Heitzman, 2011].

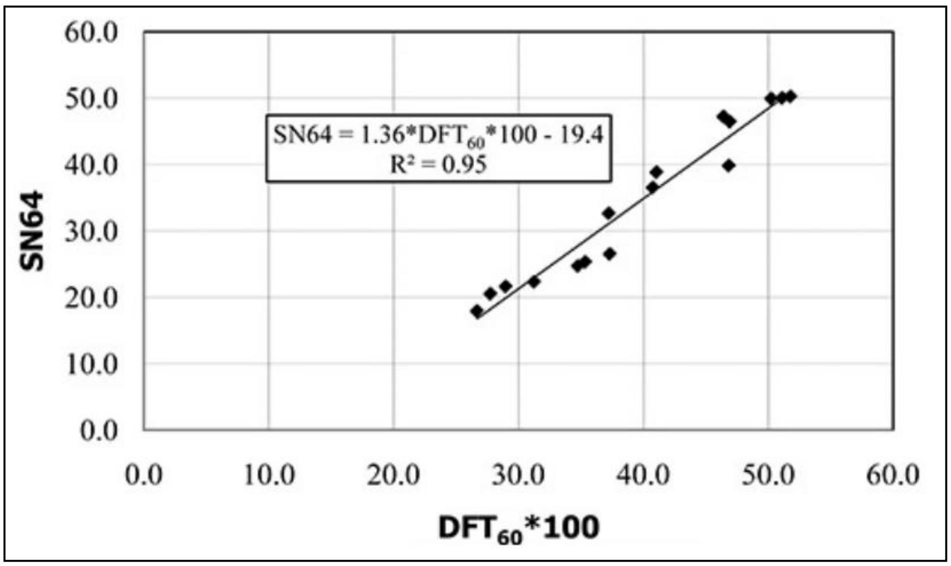

Figure 3.7: Correlation between $SN(64)_{ribbed}$ and DFT60 (metric units) [Heitzman, 2011]

3.4 MIX PROPORTIONING METHODS FOR PORTLAND CEMENT CONCRETE

Many approaches to mix proportioning have been published; most are based on the following principals:

- Fineness modulus
- Void Density
- Specific Surface
- Workability factor

Since most of the research done by ICAR at University of Texas at Austin attempted to find better methods of proportioning manufactured aggregates in concrete, only two methods for proportioning concrete were considered and evaluated during this research project. The first method is the ACI method (ACI 211), and the second is the proportioning method developed by ICAR that was made specifically for proportioning manufactured sands in concrete.

3.4.1 ACI Mixture Design Method

The ACI 211 (2002) method is based on an empirical formula that indirectly determines the amount of aggregates in a mixture. The values recommended by ACI assume that the aggregates are well graded and no guidance is given on how to blend two or more aggregates.

The ACI method relates the amount of cement needed in a mixture to strength and durability criteria in terms of minimum amount of cement and required water-to-cement ratio (*w/c*). The amount of water required increases with increasing aggregate angularity, increasing slump, decreasing maximum aggregate size, lack of air entrainment, or use of water-reducing admixtures. The volume of coarse aggregate is a function of the dry-rodded unit weight of the coarse aggregate, the fineness modulus of the fine aggregate, and the maximum aggregate size. The volume of fine aggregates depends on the amount of all other ingredients.

One of the major shortcomings of the ACI approach is that it over simplifies the proportioning process by using the fineness modulus of the sand as a factor. Research done by Young (1921), Besson (1935), and Kennedy (1940) suggest that the fineness modulus is inadequate to differentiate between sands. ACI also relates strength and durability of concrete to cement content (by specifying a minimum cement content), which is also misleading. Furthermore, ACI 211 is based on ASTM C 33 which limits the amount of microfines to a maximum of 7%.

3.4.2 ICAR Method for Proportioning Concrete

The ICAR method was originally developed by Fowler and Koehler (2007) for self consolidating concrete and was then modified by McLeroy (2009) for pavement concrete. The following are the recommended steps for designing a mixture containing MFA as:

1. Choose the aggregate system
    - Evaluate aggregate properties
    - Determine optimum grading
2. Choose the paste quantity
    - Determine minimum paste content based on the chosen combined aggregate gradation
    - Determine additional paste needed for workability based on shape and angularity of MFA
3. Choose the paste quality
    - Choose the type of Supplementary Cementitious Materials (SCM)
    - Choose air content
    - Choose w/cm

3.4.2.1 Choosing the Aggregate System

To improve the performance of concrete, it is important to properly choose aggregates based on the properties obtained from characterization tests. Each of the characterization tests has been developed to evaluate critical aggregate properties that influence concrete performance.

To achieve the highest packing density of aggregates, more than one grade of aggregate can be used. The combined gradation of coarse and fine aggregate should be evaluated using a modified 0.45 power curve. The modified 0.45 power curve should not take into account the presence of microfines since the microfines will be accounted for as part of the paste not the aggregate. The modified 0.45 power curve should go through the #200 sieve (Figure 3.8).

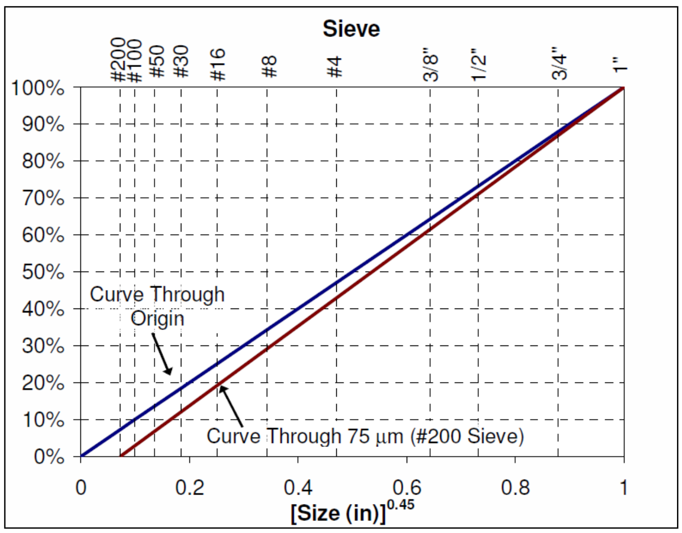

Figure 3.8: Modified 0.45 Power Curve (Fowler and Koehler 2007)

In addition to using a modified 0.45 power curve, two other methods can be used to ensure that uniform blends of aggregates are being used, these are the 8-18 grading system and the Shilstone Coarseness chart. Note that Fowler and Quiroga (2004) found that the 8-18 grading system was not suitable for evaluating aggregates with high microfine content.

After the optimal grading is determined using the modified 0.45 power curve, the dry-rodded unit weight (DRUW) of the aggregate combination should be evaluated (Tex-404-A - rodded method can be used). To ensure that highest aggregate density was obtained, multiple aggregate combinations can be tested using the modified 0.45 power curve and then by obtaining the DRUW; the combination with the highest DRUW correspond to the highest aggregate density.

After obtaining DRUW, the percent compacted voids corresponding to the chosen aggregate gradation should be determined. The percent compacted void content is determined as follows:

$$\%voids_{compacted\_agg} = \left[1 - \frac{DRUW}{(62.4)\sum_{i=1}^{n}(p_i(SG_{OD})_i)}\right] \times 100\% \qquad \text{(eq. 3.5)}$$

where DRUW is the dry-rodded unit weight of the combined aggregate (lb/ft^3), p_i is the volume of aggregate fraction i divided by the total aggregate volume, and $(SG_{OD})_i$ is the oven-dry specific gravity of aggregate fraction i.

3.4.2.2 Choosing the Paste Quantity

Figure 3.10 shows a schematic representation of aggregate in cement paste. The total volume of paste needed for concrete is equal to the volume of paste needed to fill the voids in compacted aggregates + the volume of paste needed to separate aggregate.

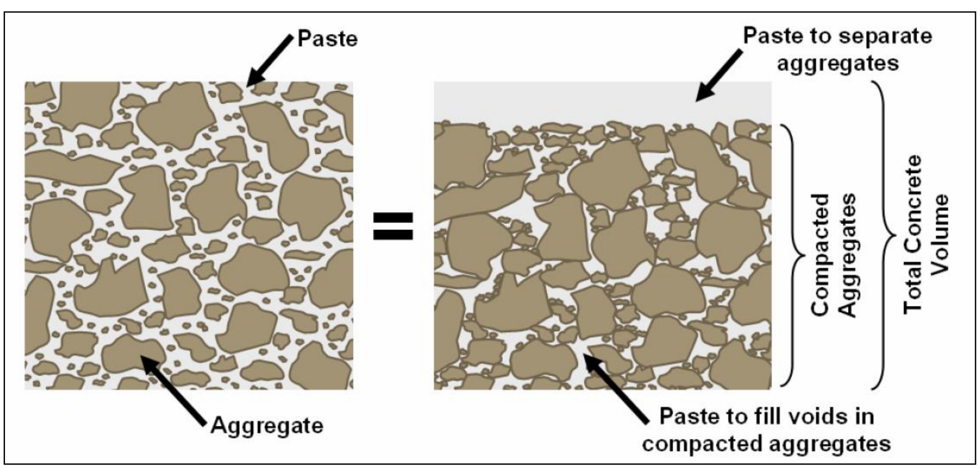

Figure 3.10: Paste Needed to Fill Voids between Aggregates [Koehler and Fowler, 2007]

$$V_{Total\ paste} = V_{paste-Voids} + V_{paste\_spacing} \qquad \text{(eq. 3.6)}$$

$V_{paste-Voids}$ corresponds to the $\%Voids_{compacted\_agg}$ calculated using DRUW. $V_{paste\_spacing}$ is related to the shape and angularity of fine aggregate. Fowler and McLeroy (2009) found that $V_{paste\_spacing}$ for a class P concrete containing high microfines content ranges from 3 to 8% paste by volume.

3.4.2.3 Choosing the Paste Quality

After the paste quantity is determined, the composition of the paste is selected to achieve the required plastic and hardened concrete properties. The paste is composed of cement, water, SCMS, air, mineral fillers (microfines present in the fine aggregates are accounted as mineral fillers), and admixtures. Table 3.3 summarizes the effect and purpose of the different paste constituents.

	Parameter	**Purpose**
Water	Water/Cement	Early-age hardened properties
	Water/Cementitious Materials	Long-term hardened properties
	Water/Powder	Workability
Powder	Cement	Strength and durability
	SCMs	Improve workability and durability, reduce heat, reduce cost
	Mineral Fillers	Improve workability, reduce cost, reduce heat
Air	Air Content	Durability

Table 3.3: Selection of Paste Composition [Fowler and Koehler 2007]

Chapter 4: Material Properties

The main goal of this research project was to evaluate the properties of fine aggregates in PCC pavement. To do so, other materials were also needed; these included cementitious materials, admixtures, and coarse aggregates. This chapter presents a list of materials used on this project as well as the results for all standard tests for fine and coarse aggregates.

4.1 CEMENTITIOUS MATERIAL AND ADMIXTURES

The cement used for all mortar and concrete mixtures was an ASTM C 150 Type I/II cement obtained from TXI Bridgeport that is used in the Dallas and Fort Worth Districts. A class F fly from Boral Material Technologies was used for the six blended sand mixtures tested for skid resistance (discussed in a later chapter). Fly ash was added to those mixtures to test blended mixtures that better represent those used in the field.

The sponsor of the project, TxDOT, required that the fine aggregates be evaluated for skid resistance using identical concrete mixture proportions. For this reason a mid-range water reducing admixture was used to facilitate the casting of concrete specimens. For making specimens that were tested for strength, modulus of elasticity, skid resistance, and shrinkage, DARACEM 55 was used. DARACEM 55 is a mid-range water-reducing admixture produced by W.R. Grace. This type of admixture is not common for slipform paving mixtures and was only used to evaluate hardened concrete properties. To evaluate concrete proportions for workability (slump), WRDA 82 was used. WRDA 82 is an ASTM C 494 Type A and D admixture produced by W.R. Grace.

4.2 FINE AGGREGATES

Fifteen fine aggregates were tested in this project. Nine of the fine aggregates were natural siliceous fine aggregates and were chosen based either on their acid insoluble residue values or because they are materials local to the Dallas and Fort Worth Districts. The following is a list of the natural sands that were evaluated:

- TXI Paradise
- Trinity Kopperl
- Chanas Eagles Nest
- Lattimore Cleburne
- Grandbury Pit #1
- Ingram Rainbow
- Lattimore Rosser
- TXI Beckett Rd.
- Colorado River Sand

Six manufactured fine aggregates were tested; some were chosen based on their mineralogy while others were selected because they are materials local to the Dallas and Fort Worth district. These MFA included:

- Hanson Servtex (limestone)
- Texas Crushed Stone-Feld Pit (limestone)
- TXI Bridgeport (limestone)
- Hanson Perch Hill (limestone)
- Capital Aggregates - Marble Falls (dolomite)
- Lattimore Materials-Stringtown (slate)

The four types of fine aggregates used are referred to as siliceous, limestone, dolomite, or slate. The lithology of those rocks was determined by the TxDOT petrographer. For the purpose of this project, evaluating the mechanical properties of the sands was determined to be more important than evaluating the exact mineral composition. To evaluate those properties ASTM or TxDOT standard tests were used. These tests included:

- Sieve analysis (Tex 401-A)
- Specific gravity and absorption (Tex 403-A & ASTM C 128)
- Dry-rodded unit weight (DRUW – Tex 404-A)
- Void content for relative shape and texture (ASTM C 1252)
- Acid insoluble residue (Tex-612-J)
- Micro-Deval (ASTM D 7428)
- Methylene blue (AASHTO TP 57-06)

All seven manufactured sands as well as one of the siliceous sands (the Colorado River Sand) were evaluated using all of the listed tests. The rest of the siliceous sands were only tested for specific gravity, absorption, acid insoluble residue, and micro-Deval. The reason that not all of the properties of the other eight siliceous sands were tested was because those properties were not needed to evaluate the hardened properties of concrete made with those sands.

4.2.1 Sieve Analysis

The sieve analysis was performed as described by Tex 401-A; the results are presented in Table 4.1. Among the six manufactured sands tested, the limestone obtained from Texas Crushed had the highest microfine content (21.9%). Hanson Servtex had the second highest microfine content (7.2%). All other manufactured sands seem to have been washed to meet ASTM C 33 limits for microfine content (less than 5% microfines for concrete subject to abrasion).

	Percent Retained						
	Colorado River Sand	Capital Aggregate Marble Falls	Texas Crushed Stone	Lattimore Stringtown	Hanson Servtex	Hanson Perch Hill	TXI Bridgeport
#4	2.8	2.0	0.8	2.5	1.4	0.5	2.1
#8	12.2	15.8	9.3	34.5	13.6	22.2	28.0
#16	16.3	25.6	21.9	27.4	18.3	30.1	29.4
#30	21.1	16.8	16.2	14.6	18.5	18.2	16.3
#50	25.5	14.9	13.0	9.6	18.0	13.2	11.6
#100	18.5	13.3	10.0	6.4	15.0	8.0	6.8
#200	2.4	8.3	6.7	2.6	7.6	4.0	2.4
Pan	0.8	2.7	21.9	2.3	7.2	3.8	3.3

Table 4.1: Sieve Analysis

4.2.2 Dry-rodded Unit Weight and Uncompacted Void Test

Results for the dry-rodded unit weight (DRUW - Tex 404-A) and the uncompacted void (ASTM C 1252) tests are shown in Table 4.2. DRUW is determined by rodding a dry sample of aggregate into a container of known volume and it is an indirect measure of aggregate shape, texture, and grading. The uncompacted void test is a measure of shape and texture and is independent of gradation (discussed in Chapter 2). The test for uncompacted voids is performed by placing a sample of re-graded sand in a funnel and allowing it to free fall into a cylinder of known volume. The mass of the uncompacted sand in the cylinder is measured, and the uncompacted void content is then computed. Lattimore Stringtown had the highest uncompacted void content as well as the lowest DRUW; this indicates that Lattimore Stringtown had the poorest shape and packing density. As expected, the Colorado River Sand had the highest DRUW value as well as the lowest void content. River sands generally have better packing densities and shapes compared to most manufactured sands.

	Colorado River Sand	Capital Aggregate Marble Falls	Texas Crushed Stone	Lattimore Stringtown	Hanson Servtex	Hanson Perch Hill	TXI Bridgeport
DRUW (lb/ft^3)	108	105.8	105.6	102.2	106.7	106.2	106.1
Uncompacted Voids (%)	39.4	46.4	47.6	48.0	43.7	44.3	44.2

Table 4.2: Dry-Rodded Unit Weight (DRUW) and Uncompacted Voids

4.2.3 Methylene Blue Test

The methylene blue test was conducted based on the procedures described in AASHTO TP 57-06 (only the aggregates passing No.200 sieve were tested). The methylene blue test indicates the presence of clay-like material in the aggregate. The results are shown in Table 4.3. All the fine aggregates were expected to perform well in concrete except for the Colorado River Sand. Although the test results indicate that that sand is marginally acceptable, the sieve analysis results indicate that the percent aggregates passing the No.200 sieve was 0.8% (Table 4.1). Thus the presence of clay in the microfines of the Colorado River Sand should not be an issue.

	Colorado River Sand	Capital Aggregate Marble Falls	Texas Crushed Stone	Lattimore Stringtown	Hanson Servtex	Hanson Perch Hill	TXI Bridgeport
MBV Value (mg/g)	10.25	4.00	5.50	3.0	6.50	3.50	3.00

Table 4.3: Methylene Blue Value (MBV)

4.2.4 Specific Gravity, Absorption, Acid Insoluble Residue, and Micro-Deval

The results for specific gravity, absorption, acid insoluble residue (discussed in section 2.1), and micro-Deval (discussed in section 2.2) for all the manufactured sands are presented in Table 4.4; the results for the siliceous sands are presented in Table 4.5.

	Lattimore Stringtown	Capital Aggregate Marble Falls	Texas Crushed Stone	Hanson Servtex	Hanson Perch Hill	TXI Bridgeport
Lithology	Slate	Dolomite	Limestone	Limestone	Limestone	Limestone
SG_{SSD}	2.54	2.78	2.55	2.57	2.63	2.60
SG_{OD}	2.52	2.77	2.48	2.51	2.58	2.55
Absorption(%)	0.84	0.38	2.57	2.29	2.04	2.22
Acid Insoluble Residue (AIR %)	77	2.3	0	1.2	6.7	1
Micro-Deval (% Loss)	8.9	11.6	21.8	26.8	22.8	19.1

Table 4.4: Specific Gravity, Absorption, Acid Insoluble Residue, and Micro-Deval Percent Loss for MFA

	TXI Paradise	Colorado River Sand	TXI Beckett Rd.	Eagle's Nest	Ingram Rainbow	Lattimore Cleburne	Lattimore Rosser	Trinnity Kopperl	Granbury Pit #1
Lithology	Siliceous	Siliceous	Siliceous	Siliceous	Siliceous	Siliceous	Siliceous	Siliceous	Siliceous
SG_{SSD}	2.65	2.60	2.67	2.63	2.63	2.63	2.67	2.64	2.64
SG_{OD}	2.64	2.58	2.64	2.62	2.62	2.62	2.65	2.63	2.61
Absorption(%)	0.62	0.45	0.77	0.6	0.52	0.58	0.79	0.68	1.11
Acid Insoluble Residue (AIR %)	74.4	84.5	77	95.3	85.9	72.6	73.6	76.8	98
Micro-Deval (% Loss)	8.2	7.7	6.5	7.6	8.3	7.1	6.4	6.8	3.5

Table 4.5: Specific Gravity, Absorption, Acid Insoluble Residue, and Micro-Deval Percent Loss for Siliceous Sands

Three specific gravity and absorption tests were performed on each aggregate, but only one value was reported in Table 4.5 (the other values were used to check that the reported value was in the same range). Two AIR tests were performed on each aggregate, the average standard deviation for the AIR test was 3.1%. As for the micro-Deval test, at least two tests were performed for each aggregate and the average standard deviation between tests was 0.5%.

The values obtained for specific gravity, absorption, acid insoluble residue, and micro-Deval for all the siliceous sands are not significantly different. The absorption values for the limestone sands were higher than for all of the other sands tested. The dolomite and slate sands had absorption values closer to the values of the siliceous sands. All carbonate aggregates failed the acid insoluble residue test (did not meet the 60% limit). Compared to the other carbonate aggregates, the Capital Marble Falls (dolomite) had a lower micro-Deval percent loss. The micro-Deval percent loss for the siliceous sands ranged from 3.5 to 8.2%, while the micro-Deval percent loss for limestone sands ranged from 19.1 to 26.8%.

4.3 COARSE AGGREGATES

Two coarse aggregates were used in this project. Both were TxDOT grade 4 limestone obtained from TXI Bridgeport and Hanson Perch Hill (note that Table 3 of Item 421 of the TxDOT Manual defines aggregate grades). The reason two coarse aggregates were used was not because differences in performance were expected, but because the sponsor of the project wanted the research to include materials from two different producers.

The specific gravity and absorption of the coarse aggregates was tested using the method described in ASTM C 127; the results are shown in Table 4.6. Tex-401-A was used to evaluate the grading of the coarse aggregates; results are shown in Table 4.7.

	Hanson Perch Hill (Coarse Aggregate)	TXI Bridgeport (Coarse Aggregate)
Lithology	Limestone	Limestone
SG_{SSD}	2.67	2.65
Absorption (%)	0.69	1.09

Table 4.6: Specific Gravity and Absorption for Coarse Aggregates

	Percent Retained	
	Hanson Perch Hill (Coarse Aggregate)	**TXI Bridgeport (Coarse Aggregate)**
1 ½ in.	0	0
1 in.	2.1	2.4
¾ in.	11.9	13.8
½ in.	21.8	29.5
3/8 in.	11.2	16.9
#4	39.8	28.2
#8	11.8	7.0
Pan	1.3	1.2

Table 4.7: Sieve Analysis for Coarse Aggregates

4.4 CONCLUSIONS

The material properties listed in this chapter were determined before the concrete testing started. Those properties were used to proportion all the mortar and concrete mixtures tested in this research. The aggregate properties were also compared to concrete performance test results; this will be discussed in a later chapter.

Chapter 5: Non-Standard Micro-Deval Aggregate Testing

The reason aggregate properties are determined is to evaluate their potential performance in concrete. For an aggregate test to be viable, it has to correlate well with concrete laboratory or field performance. It is simpler and faster to test an aggregate specimen than to test a concrete made from this aggregate; that is why aggregate tests are preferred tools for predicting performance. This chapter will discuss an attempt to find better or improved methods for testing fine aggregates using the micro-Deval apparatus.

5.1 Testing Fine Aggregates Using the Micro-Deval apparatus

The Standard ASTM D 7428 micro-Deval test for fine aggregates consists of placing a 500g of a graded sand sample, 1250g of 10mm steel ball bearing, and 750ml of water in the micro-Deval jar (Figure 5.1). The jar is put into rotation for 15 minutes at 100 rpm. After the sample is run in the micro-Deval device, it is washed over a No. 200 sieve and the retained sample is oven dried (the washed fines passing the No. 200 are discarded). The percent loss in mass is computed from the oven dried sample.

Figure 5.1: Micro-Deval Jar

At the beginning of this research project, it was not clear whether or not the current settings Including test time, sample size, and weight of ball bearings, for the micro-Deval test were adequate to differentiate between aggregates for skid resistance. For this reason an investigation was made to find out if changing the micro-Deval settings could result in an improved test. Instead of running the micro-Deval for only 15 minutes, the test was run for 15, 60, and 120 minutes (all other settings were not changed). Note that only one test was performed for each aggregate at 60 and 120 minutes. Four sources of sands were tested:

- Colorado River Sand: a siliceous aggregate that meets the acid insoluble residue limits and that is expected to perform well.
- Capital Marble Falls: a dolomite that is known to be harder than other limestone carbonate aggregates but that fails the acid insoluble residue test.
- Hanson Servtex: a limestone fine aggregate that fails acid insoluble residue and is not expected to perform well in PCC pavements.
- Texas crushed stone: a soft limestone that is expected to have very poor skid performance in PCC.

A summary of the results is shown in Figure 5.2. The limestone sands had the highest percent loss at all testing times. The percent loss for the Colorado River Sand was the lowest at 15 and 60 minutes, while Capital Marble Falls had the lowest percent loss at 120 minutes. At 15 and 60 minutes, Capital Marble Falls had a percent loss that was higher than the siliceous sand and lower than the two limestone sands.

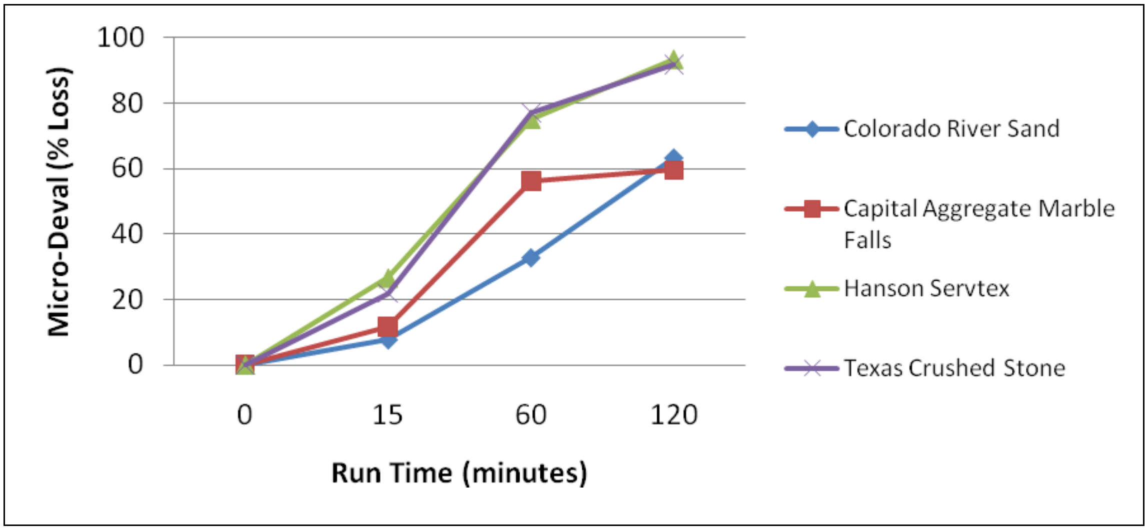

Figure 5.2: Varying Run Time for Micro-Deval Fine Aggregate Testing

When the micro-Deval test was run for 60 and 120 minutes, there was a huge reduction in the quantity of aggregates remaining (Figure 5.3). Most of the sand left after the test consisted of the larger sized fine aggregate (No. 8 and No. 16).

Figure 5.3: Hanson Servtex Before and After Micro-Deval (120 Minutes Run time)

The micro-Deval test is considered to be an abrasion test for coarse aggregates. For fine aggregates, the difference in size between the steel ball bearings and the fine aggregates makes the micro-Deval seem more of a crushing test. Figure 5.4 shows the difference in size between fine aggregates, coarse aggregates, and the 10mm steel ball bearings used for the micro-Deval test.

Figure 5.4: Fine and Coarse Aggregate Sizes Compared to 10mm Ball Bearings

When coarse aggregates are tested in the micro-Deval, there is no major reduction in the size of the aggregates. Coarse aggregates tested in the micro-Deval are smoother and less angular than they originally were; this indicates that coarse aggregates are being abraded and not crushed.

To verify that fine aggregates are being crushed in the micro-Deval rather than just abraded, a sieve analysis was performed on all the fine aggregates tested (only one test was performed for each aggregate). Figures 5.5, 5.6, and 5.7 show the percent change in fine aggregate gradation after testing the aggregates in the micro-Deval for 15, 60, and 120 minutes. Compared to the original gradation of the sample placed in the micro-Deval, Figure 5.5 shows that there was a reduction in the percent of aggregates retained on the No. 30 sieve. There was also an increase in the percent of aggregates retained on the No. 200 sieve. When aggregates were tested for 60 minutes, there was a significant reduction in the percent of aggregates retained on the No. 30, No. 50, and No. 100 (Figure 5.6). Texas crushed stone and Hanson Servtex (the limestone sands), experienced a loss of No. 16 retained aggregates that was larger than the other two aggregates. In general, there seems to be a lower reduction in the percentage of aggregates retained on the No. 8 and No. 16 sieves; this might be attributed to the ball bearings having more of an abrasion effect on the larger sizes of fine aggregate. When the test was run for 120 minutes, the percentage of aggregates passing the No. 16 sieve was greatly reduced (Figure 5.7). The limestone sands experienced a loss of all sizes of fine aggregates including fine aggregates retained on the No. 8 and No. 16 sieves.

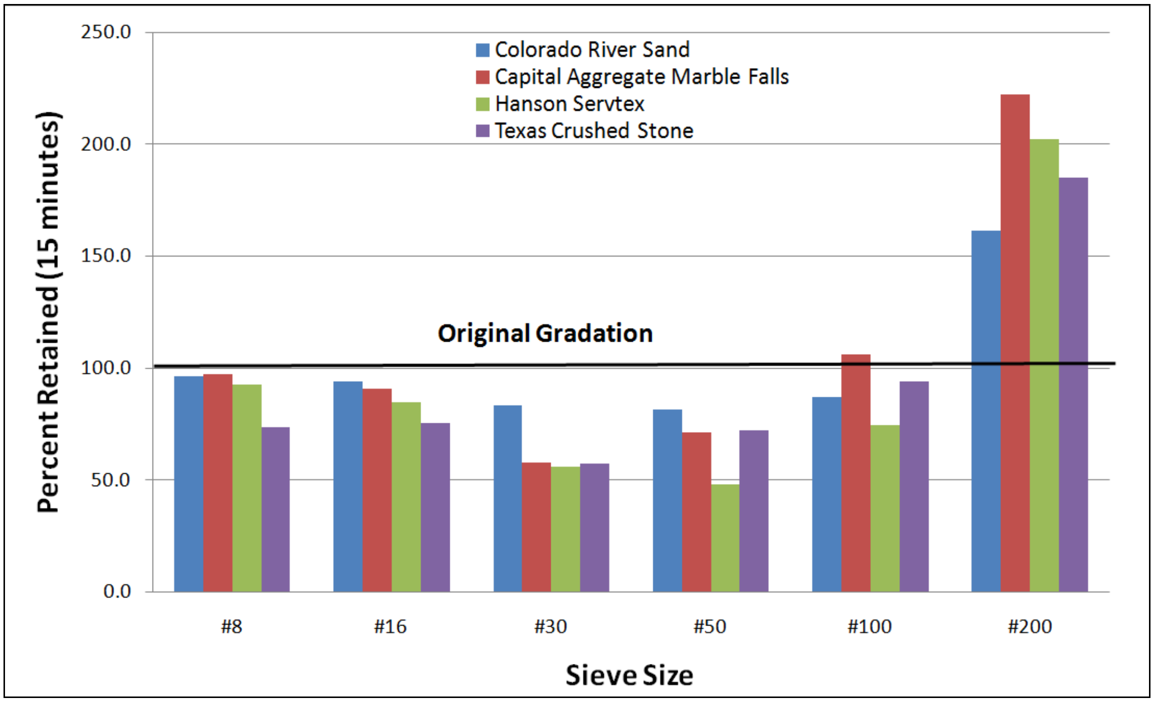

Figure 5.5: Percent Change in Gradation After a 15 Minutes in the Micro-Deval Test

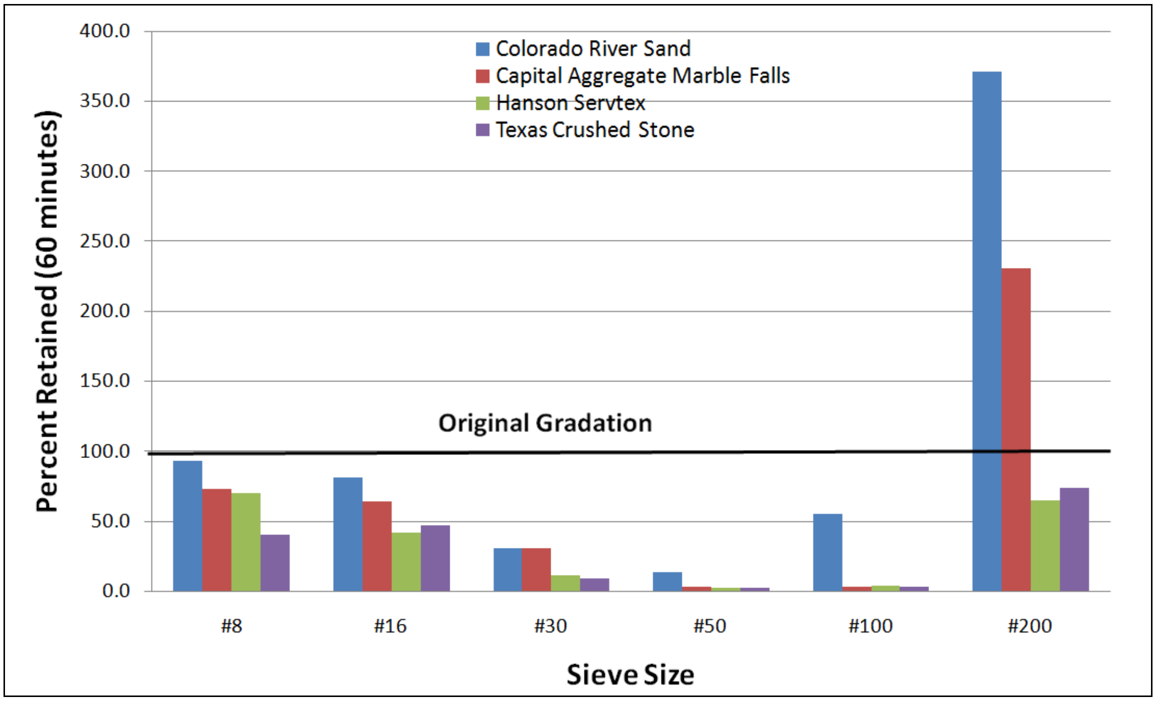

Figure 5.6: Percent Change in Gradation After a 60 Minutes in the Micro-Deval Test

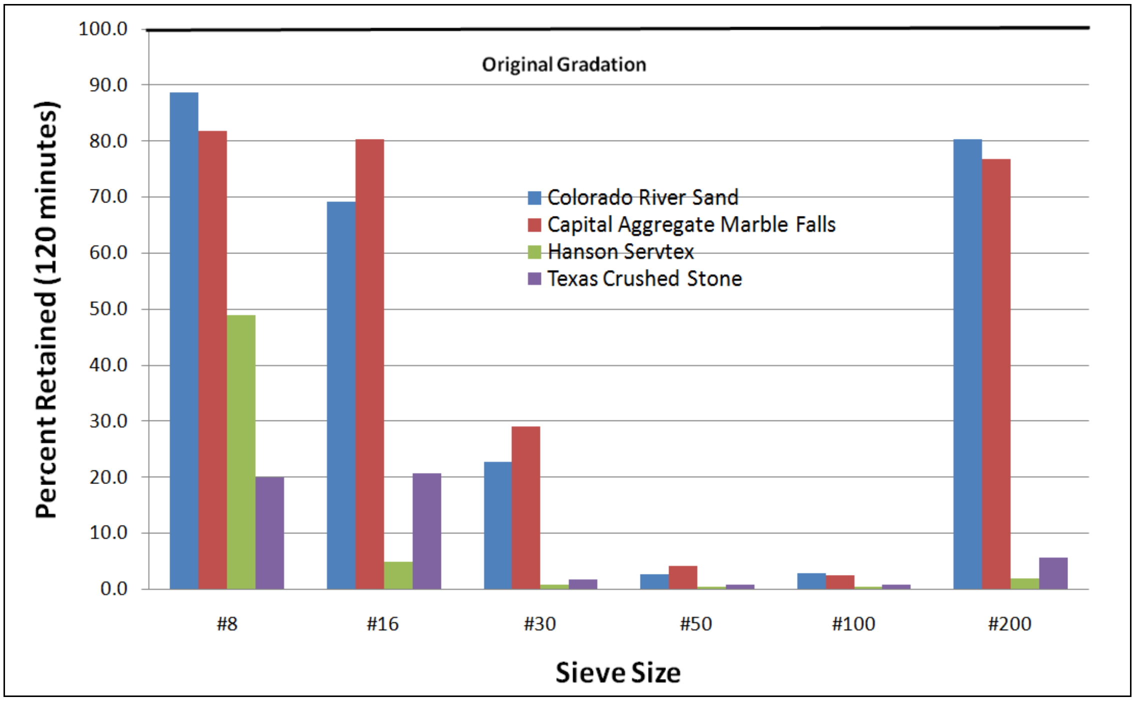

Figure 5.7: Percent Change in Gradation After a 120 Minutes in the Micro-Deval Test

The test results for micro-Deval percent loss and gradation change indicate that:

- Capital Marble Falls (dolomite) had a percent loss close to that of the Colorado River Sand when the tests are run for 15 or 120 minutes. At 60 minutes, Capital Marble Falls was about half way between the siliceous sand and the limestone sands.
- Fine aggregates are crushed and abraded in the micro-Deval; larger sizes of sand get abraded, while the smaller sizes get crushed. Crushing becomes the dominating cause of lose of materials when the micro-Deval test was run for a longer time period.

Polishing of aggregates in PCC pavement is believed to occur due to abrasion of fine aggregates caused by traffic. It would therefore be more appropriate to have an aggregate test that can simulate abrasion rather than crushing of fine aggregate. To achieve that, the following changes to the micro-Deval test were considered:

1. Using smaller steel ball bearings: Reducing the size of the ball bearings to match the fine aggregate size might be a good idea in theory but it is not a practical solution. An attempt was made to use 3-mm ball bearings. Due to their size, the 3-mm ball bearings were hard to recover and that made such a test not practical.
2. Testing of a coarse aggregate obtained from the same source as the fine aggregate: At the beginning this sounded like a good idea, but after the test was attempted several problems were identified. The shape and texture of coarse aggregates play a big role in abrasion loss; this is not necessarily true for fine aggregates used in PCC. The aggregates being tested also do not represent the aggregates being used (same source but different sizes).
3. Testing of mortar specimen: Among the three ideas considered this was the most promising. Details of the mortar testing using micro-Deval are described in the next section of this chapter.

5.2 TESTING MORTAR ABRASION USING THE MICRO-DEVAL APPARATUS

The test described in this section does not follow any known standards for testing mortars or aggregates. The testing done on mortar using the micro-Deval was an attempt to find a better test method for evaluating fine aggregates for skid resistance by testing mortar specimens rather than fine aggregate specimens. The reason a micro-Deval mortar test would be better than the current fine aggregate test is because the larger size of the mortar specimen will allow the sands in the mortar to be abraded rather than crushed. Moreover, some blended sands are blended before the individual properties of each of the sands are tested. Because aggregates tested in the micro-Deval are washed over a No.200 sieve and then graded, testing blended sands using the fine aggregate micro-Deval test will probably result in a lower percent loss for those blended sands and that will result in less conservative percent loss values (softer manufactured sands often have higher microfine content).

Mortar specimens measuring 1 ½ in. wide and ¾ in. deep were cast using the same four fine aggregates described in the first section of this chapter. The mold used was a silicone mold made for baking brownies. For this reason the mortar specimens will also be referred to as "mortar brownies". To test the mortar brownies, the following procedures were followed:

- The mortar brownies were placed in water and cured for seven days.
- After seven days of curing the mortar brownies were oven dried for twenty-four hours.
- The oven-dried weight of each of the mortar brownies was measured and recorded.
- Seven or nine mortar brownies were placed in the micro-Deval jar along with 1200g or 3000g of ball bearings and 2000ml of water.
- The jars containing the specimen were run in the micro-Deval for 2 hours.

- The abraded mortar brownies were removed from the micro-Deval jar by hand and washed (no sieves were required).
- The abraded specimens were oven dried for 24 hours and their oven-dried weight was measured and then recorded.
- The percent change in mass was computed using the recorded oven-dried weights.

The results of the percent loss in weight for specimens made with different sands at different water-to-cement ratios are shown in Figures 5.8 and 5.9. Note that only one batch per mixture was made for each test. The results in Figure 5.8 show the percent loss in weight for tests where nine specimens of mortar were tested with 3000g of ball bearings. The results in Figure 5.9 show the percent loss in weight for tests where seven specimens of mortar were tested with 1200g of ball bearings.

The results obtained for percent loss in weight were not as expected; the Colorado River Sand had the highest percent loss. All the other carbonate aggregates generally had lower percent loss at the different water-to-cement ratios. The percent loss increased when the water-to-cement ratio was increased from 0.32 to 0.6. The percent loss also increased when more steel ball bearings were used.

The results of the percent weight loss of the mortar brownies does not correlate well with the expected field performance. The percent weight loss measured in this test better represents a loss in macro-texture of mortar and not a loss of micro-texture. In other words, it is a measure of how much mortar is getting abraded and not a measure of the polishing of fine aggregates. The abraded mortar specimens made with the Colorado River Sand at w/c=0.6 had rougher textures compared to all other abraded specimens (Figure 5.10). The mortar specimens made with the same sand at w/c=0.32 seemed to have less texture.

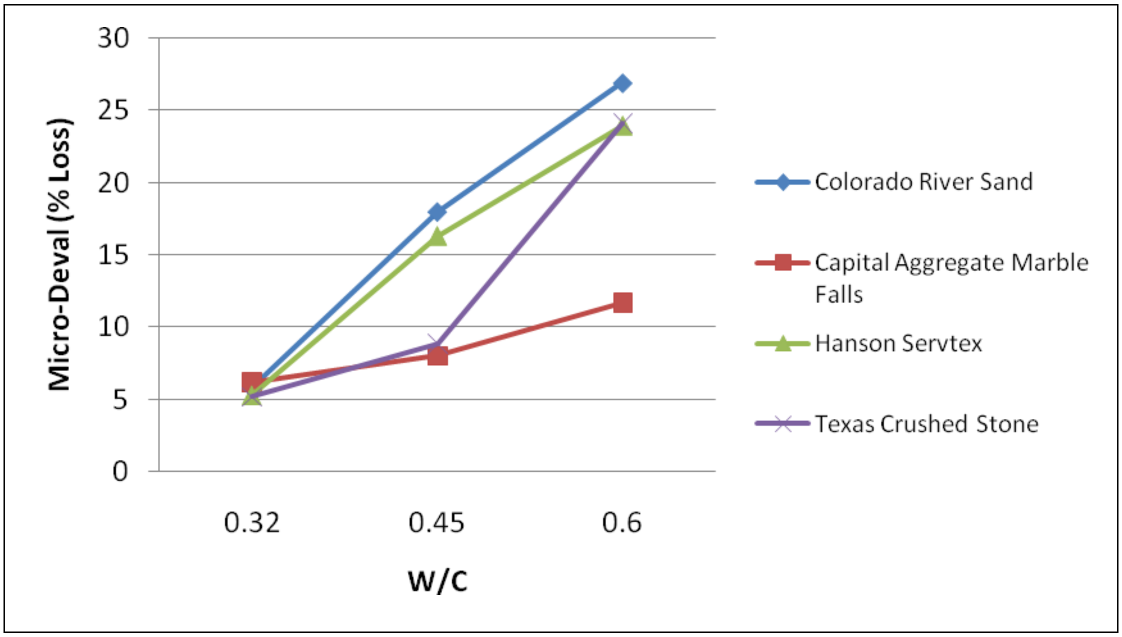

Figure 5.8: 9 Abrasion of Mortar Specimens Using 3000g of Ball Bearings

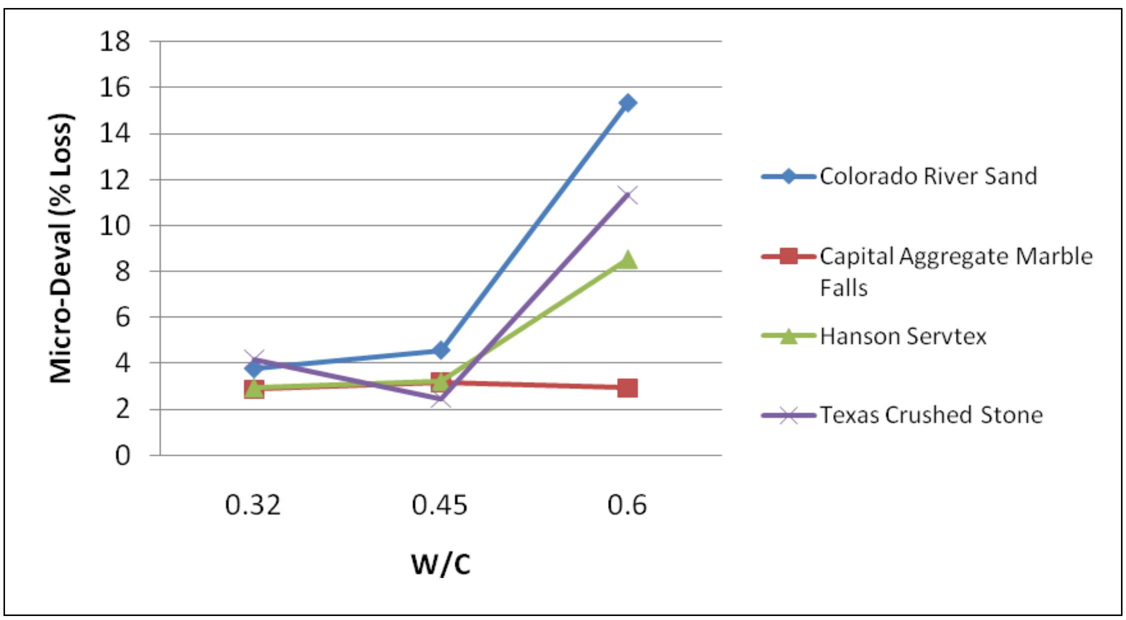

Figure 5.9: 7 Abrasion of Mortar Specimen Using 1200g of Ball Bearings

Figure 5.10: Mortar Brownies Made with Siliceous Sand at a water-to-cement ratio of 0.45 and 0.6

The difference between the texture of the abraded mortar brownies made with carbonate sands and the siliceous sand was palpable, but that was not enough to evaluate their texture. To quantify the difference in texture, AIMS was used. AIMS is capable of evaluating the shape and texture of coarse aggregates. For the purpose of evaluating texture created by fine aggregates in mortar, the mortar brownies were tested for texture the same way coarse aggregates would be. The older model of AIMS was used for this purpose (Figure 5.11). The texture index values for three specimens made with siliceous sand and three other specimens made with limestone sand at a w/c=0.6 were measured using AIMS. The results illustrated in Figure 5.12 show that AIMS is able to differentiate between the textures of the mortar brownies made with different sands (each bar represents the texture of one brownie).

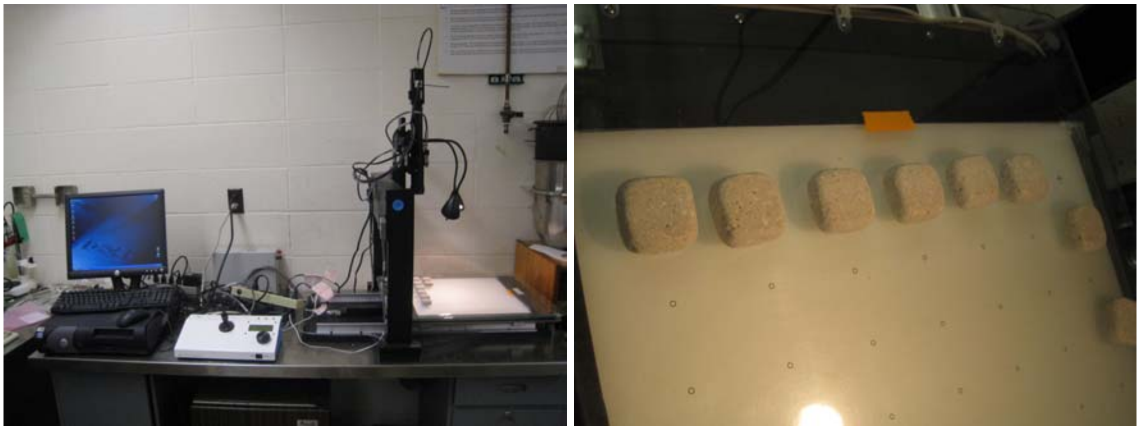

Figure 5.11: Mortar Specimen Tested Using Original AIMS Apparatus

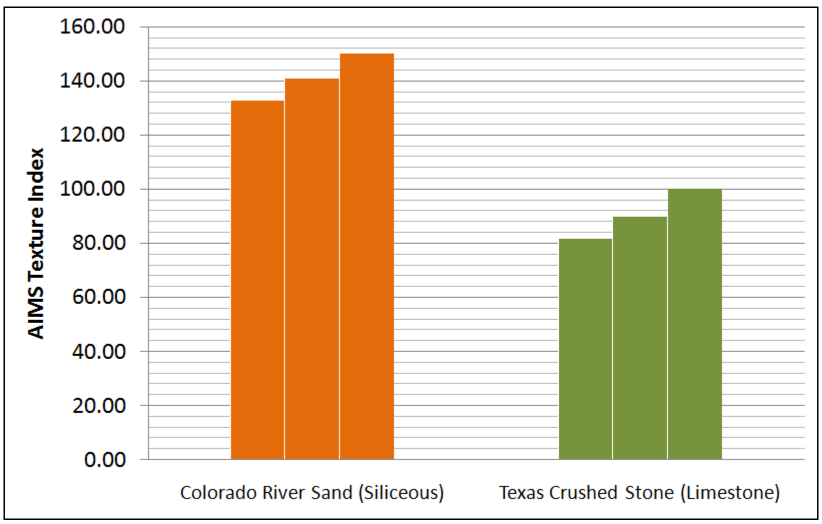

Figure 5.12: AIMS Texture Index Results Using the Original AIMS Device

The results obtained so far seemed promising but the test still needed to be improved. Based on the results obtained from the initial tests, a higher water-to-cement ratio was needed to expose the sand particles. It was also important to increase the sand content in the mortar because the aim of the test was to evaluate the texture created by that sand. Table 5.1 shows the volumetric mixture proportions used to make the second batch of mortar brownies.

Material Volume (%)		
Cement	**Water**	**Sand**
11.55	25.46	62.99

Table 5.1: Mixture Proportion Used for Mortar Mixtures

To test the second batch of brownies, the following procedures were followed:

- The mortar specimens were made using the same 1 ½ in. wide and ¾ in. molds
- The brownies were placed in containers and cured in water for 7 days.
- For each of the sands, six brownies were tested in the micro-Deval with 2500g of steel ball bearings and 2000ml of water for 1 hour.
- The brownies were removed from the micro-Deval jar and placed in the oven for one hour.
- AIMS was then used to evaluate the texture of the finished surface of the mortar brownies; the finished surface is also the wide surface.

Three sands were tested; these included the Colorado River Sand (siliceous), Texas Crushed Stone (limestone), and Lattimore Stringtown (Slate). Instead of using the AIMS machine shown in Figure 5.11, a newer version of AIMS was used (Figure 5.13). The main difference between the two devices is that the newer version of AIMS is not influenced by external sources of light. Such a device is capable of measuring more consistent and repeatable texture index values.

Figure 5.13: New AIMS Apparatus (AIMS 2.0)

Results of the AIMS texture index values are shown in Figure 5.14. Each bar in Figure 5.14 represents the texture of one mortar brownie. Compared to Figure 5.12, a more significant difference between the texture of the siliceous and limestone sand was obtained. This occurred because a higher sand content was used (the higher sand content created more texture). The higher texture index values obtained on the brownies made with Lattimore Stringtown do not seem to only be attributed to the sand, but also to presence of air voids on the surface of the abraded brownie mortar (Figure 5.15 – the mortar brownie on the left). The mortar brownies made with Lattimore Stringtown have more air voids than the brownies made with the Colorado River Sand (Figure 5.15, the brownie in the middle) and more air voids than the brownies made with Texas Crushed Stone (Figure 5.15, the brownie on the right). The air voids problem seemed to only be prominent when Lattimore Stringtown was used. Mortar brownies made with Hanson Servtex and Capital Marble Falls were also cast and tested in the micro-Deval but none of those had significant air void content after being abraded.

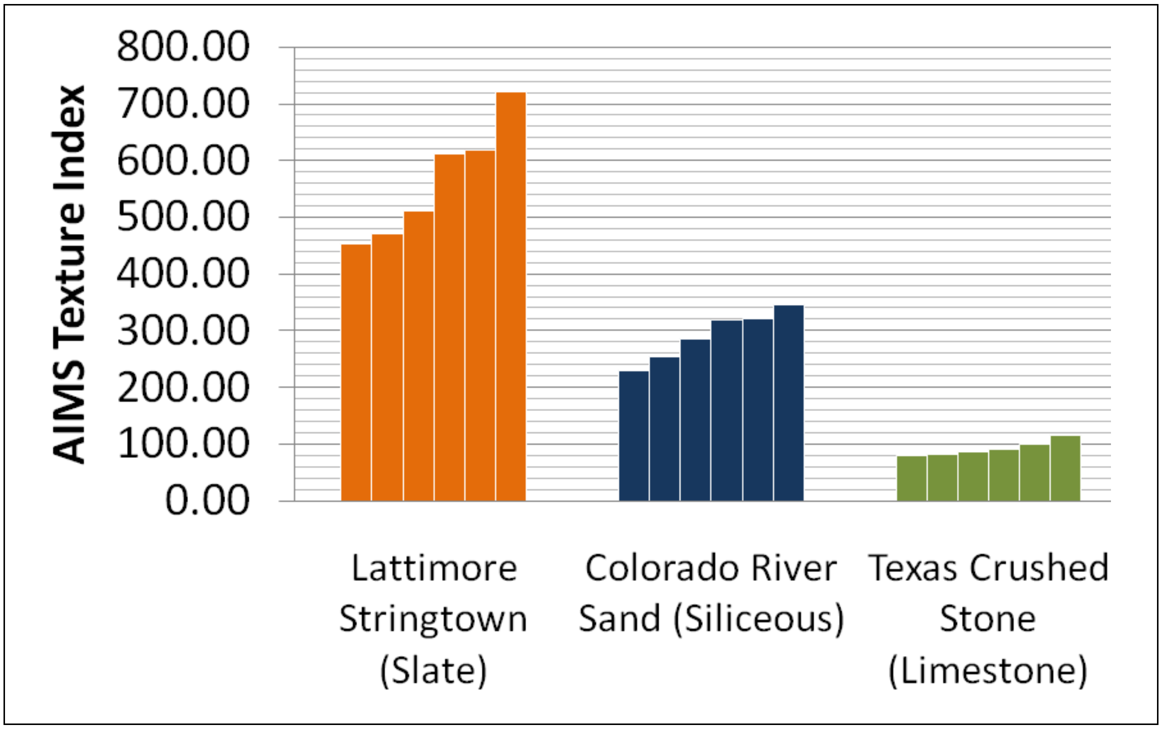

Figure 5.14: AIMS Texture Index Results Using the New AIMS Device

Figure 5.15: Mortar Specimen Tested for Texture (from left to right Lattimore Stringtown, Colorado River Sand, and Texas Crushed Stone)

A third batch of mortar brownies was made using the exact mixture proportions used for the second batch. To reduce the air void content, 9ml of alcohol was added per 1000ml of mortar used to make the brownies. The molds were also vibrated using a table vibrator. The abraded finished surface obtained from those mixtures had less exposed aggregates (Figure 5.16). Although adding alcohol and vibrating the molds seemed to have reduced the air void content for the mortar brownies made with Lattimore Stringtown, less aggregate was exposed. This makes evaluating the texture created by fine aggregate not possible because not enough fine aggregates are exposed by abrasion.

Figure 5.16: Abraded Lattimore Stringtown and Colorado River Sand Mortar Specimens

Testing mortars in the micro-Deval might be the best way of evaluating polish resistant aggregates because it better simulates abrasion of fine aggregates in concrete. The problem that still remains, however, is finding the best proportions of materials and a casting method that would result in an air-void free abraded surface with exposed fine aggregates.

5.3 CONCLUSIONS

Although the additional work done on the micro-Deval test did not result in finding a better method to test fine aggregates, it did help to better understand what aggregate properties the micro-Deval test was evaluating. The 15-minute run time adopted by ASTM seems to be better than the longer times attempted, because when the micro-Deval was run for longer periods of time more crushing of fine aggregates occurred. The procedures to make and test the mortar specimen in the micro-Deval have not yet been optimized to the extent where reliable and repeatable results could be obtained. However, the test is very promising because it could be used to not only evaluate individual sources of sand, but to also more accurately evaluate pre-blended sands.

Since more research needs to be done to improve the micro-Deval test for mortars, the results obtained from this chapter will not be correlated with results obtained from concrete tests. The only micro-Deval test results that will be compared to concrete results are the results that were presented in chapter 4.

Chapter 6: Evaluating the Shape of MFA

During the manufacturing process of aggregates, the type of crusher and the crushing speed influences the shape, texture, and grading of the manufactured sand product. The shape of manufactured sands can be improved if the crushing operation is optimized to produce better shaped aggregates. Producing better shaped fine aggregates would encourage the use of more manufactured fine aggregate because less workability and finishability problems would be encountered.

To investigate how much improvement in shape could be obtained by optimizing the crushing operation, two materials were sent to the Metso Mineral Research and Test Center (MRTC) in Milwaukee. The two materials sent to MRTC were rocks obtained from the Lattimore Stringtown and Hanson Perch Hill aggregate pits. MRTC crushed each of those rocks using a Barmac B3000 VSI crusher at three different speeds.

In Chapter 4, the dry-rodded unit weight test (Tex 404-A) as well as the uncompacted void test (ASTM C 1252) were used to compare the shape, texture, and packing densities of all the manufactured sands. In this chapter the shape and texture of fine aggregates were tested using the uncompacted void test, AIMS, and a Mortar flow test (ASTM C 1437). The nine aggregates that were evaluated included:

- Colorado River Sand: this is the control well-shaped siliceous sand.
- Lattimore Stringtown: is the sand that was crushed by Lattimore Materials and obtained from the Lattimore Stringtown pit.
- Hanson Perch Hill is the sand that was crushed by Hanson Materials and obtained from the Hanson Perch Hill pit.
- Lattimore Stringtown (Metso 55 m/s): is the sand that was produced by MRTC by crushing a rock obtained from Lattimore Stringtown at a crushing speed of 55 m/s.

- Lattimore Stringtown (Metso 60 m/s): is the sand that was produced by MRTC by crushing a rock obtained from Lattimore Stringtown at a crushing speed of 60 m/s.
- Lattimore Stringtown (Metso 65 m/s): is the sand that was produced by MRTC by crushing a rock obtained from Lattimore Stringtown at a crushing speed of 65 m/s.
- Hanson Perch Hill (Metso 50 m/s): is the sand that was produced by MRTC by crushing a rock obtained from Hanson Perch Hill at a crushing speed of 50 m/s.
- Hanson Perch Hill (Metso 55 m/s): is the sand that was produced by MRTC by crushing a rock obtained from Hanson Perch Hill at a crushing speed of 55 m/s
- Hanson Perch Hill (Metso 60 m/s): is the sand that was produced by MRTC by crushing a rock obtained from Hanson Perch Hill at a crushing speed of 60 m/s

Some of the aggregates tested are shown in Figures 6.1, 6.2, and 6.3. The Colorado River Sand has a shape that is better than all the manufactured sand. Hanson Perch Hill is not as angular as Lattimore Stringtown (Figure 6.1). Hanson Perch Hill and Hanson Perch Hill (Metso 60 m/s) appear to be very similar (Figure 6.2). Figure 6.3 shows that Lattimore Stringtown has more flat and elongated particles compared to Lattimore Stringtown (Metso 65 m/s).

Figure 6.1: Aggregates Retained on the No. 8 sieve; left-to-right, Colorado River Sand, Hanson Perch Hill, and Lattimore Stringtown

Figure 6.2: Aggregates Retained on the No. 8 sieve; left-to-right, Colorado River Sand, Hanson Perch Hill (Metso 60 m/s), and Hanson Perch Hill

Figure 6.3: Aggregates Retained on the No. 8 sieve; left-to-right, Colorado River Sand, Lattimore Stringtown (Metso 65 m/s), and Lattimore Stringtown

6.1 Uncompacted Void Test Results

The nine aggregates were evaluated using the uncompacted void test (ASTM C 1252). The uncompacted void test (discussed in 4.2.2) is an indirect test that evaluates shape and texture by comparing the packing densities of fine aggregates. The results of the uncompacted void test for the Hanson Perch Hill aggregate are shown in Figure 6.4. Note that each test was performed twice, and the average standard deviation was 0.09%. The Colorado River Sand had the lowest percent of uncompacted void; this indicates that it had a better shape. Hanson Perch Hill (Metso 60 m/s) had the second lowest percent of uncompacted voids. The uncompacted void percent seems to decrease as the crusher speed for the Metso aggregates increases. This indicates that by increasing the crusher speed, Metso was able to produce a better shaped aggregate.

Figure 6.5 shows the results for the Lattimore Stringtown aggregate. All aggregates crushed by Metso had better packing densities than the aggregate crushed by Lattimore Materials. Increasing the crusher speed for the Lattimore Stringtown aggregate did not improve the aggregate's packing density since the lowest packing density for the Lattimore aggregates was obtained at 55 m/s.

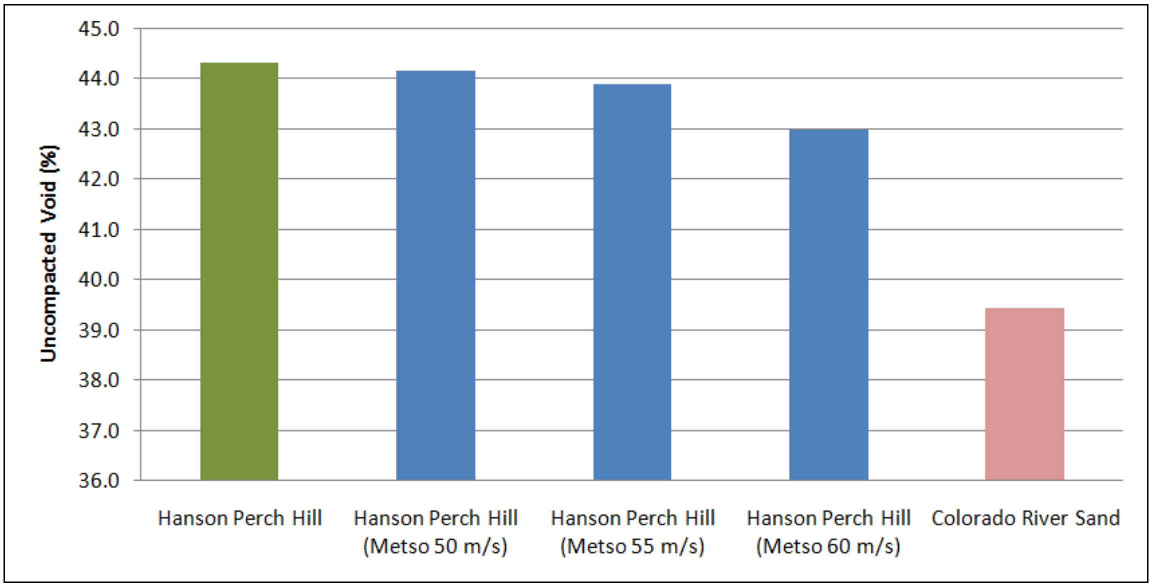

Figure 6.4: Uncompacted Void Test for the Hanson Perch Hill Aggregates

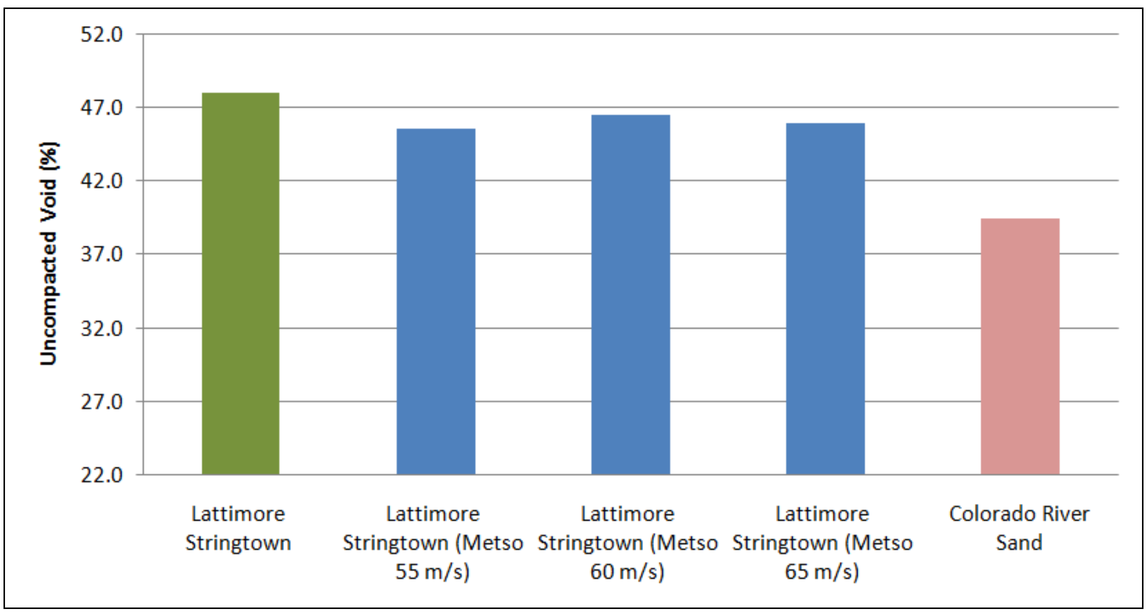

Figure 6.5: Uncompacted Void Test for the Lattimore Stringtown Aggregates

6.2 AIMS Results

The AIMS device (shown in Chapter 5, Figure 5.12) was used to evaluate the shape and angularity of the aggregates. The sizes tested were aggregates retained on No. 8, No. 16, No. 30, and No. 50. Each tested sample consisted of at least one hundred particles from each size and each sample was only tested once. AIMS evaluates the shape of fine aggregates by using a 2D form index. The 2D form index scale ranges from 1 to 20; the lower the form index the more equidimensional a particle is. Figure 6.6 shows the cumulative percentage of fine aggregates having a shape factor less than 6 for the Hanson Perch Hill aggregates. The Colorado River Sand had the highest percentage of aggregates with a 2D form index that is less than 6; this indicates that it had the best shape. The Hanson Perch aggregates produced by Metso had higher values than the original Perch Hill aggregate produced by Hanson.

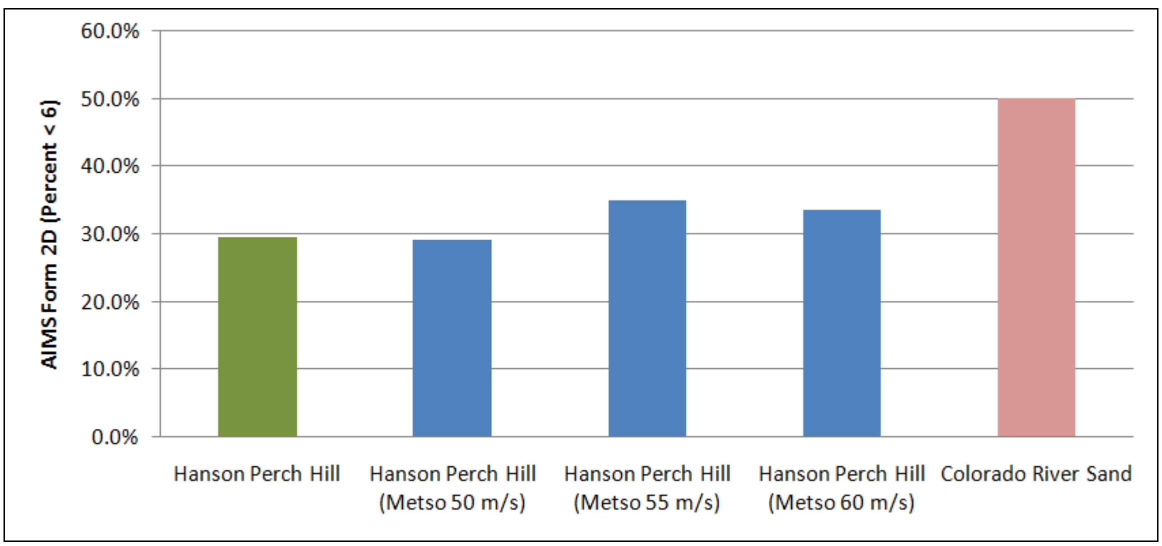

Figure 6.6: Cumulative 2D Form Index for the Hanson Perch Hill Aggregate

Figure 6.7 shows the cumulative percentage of fine aggregates having a shape factor less than 6 for the Lattimore Stringtown aggregates. All the aggregates produced by Metso had better shapes than the aggregate produced by Lattimore. Compared to the Perch Hill aggregates in Figure 6.6, the Lattimore Stringtown aggregates in Figure 6.7 had significantly lower percentages of aggregates with a shape factor less than 6. The better shape of the Perch Hill aggregate can be attributed to Perch Hill's mineralogy; better shaped aggregates can be produced with limestone aggregates.

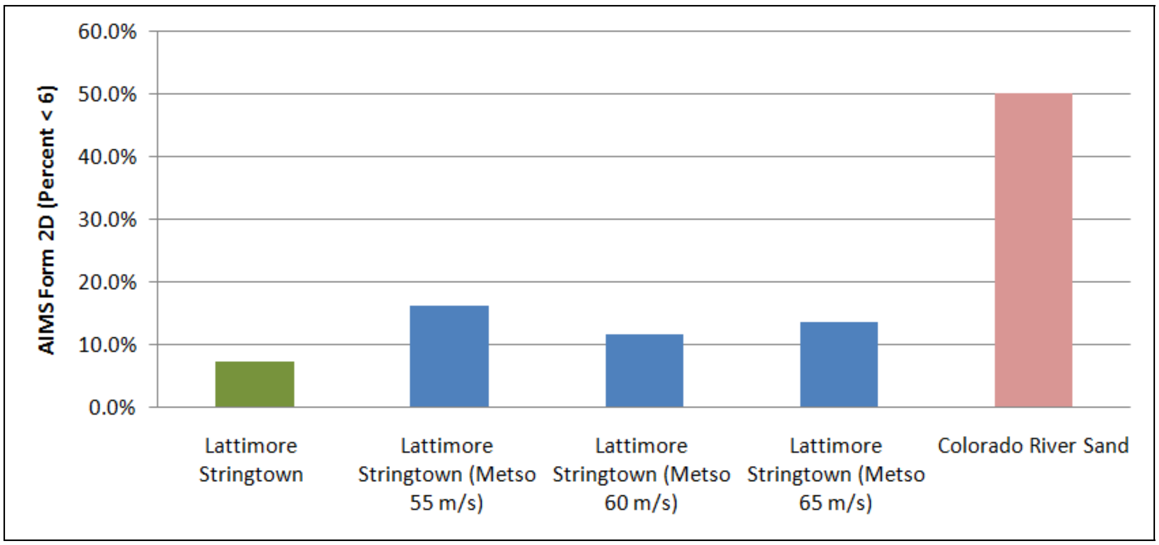

Figure 6.7: Cumulative 2D Form Index for the Lattimore Stringtown Aggregate

AIMS can also evaluate the angularity of fine aggregates. The scale used ranges from 0 to 10000; 0 indicates the presence of well round aggregates, and 10000 indicates the presence of highly angular aggregates. Figure 6.8 shows the cumulative percentage of fine aggregates having an angularity factor less than 3300 for the Hanson Perch Hill aggregates. Less angular aggregates were produced by Metso when the crusher speed was increased from 50 to 60 m/s. For the Hanson Perch Hill (60 m/s) the percent of aggregates with an angularity index less than 3300 was almost equal to that of the Colorado River Sand.

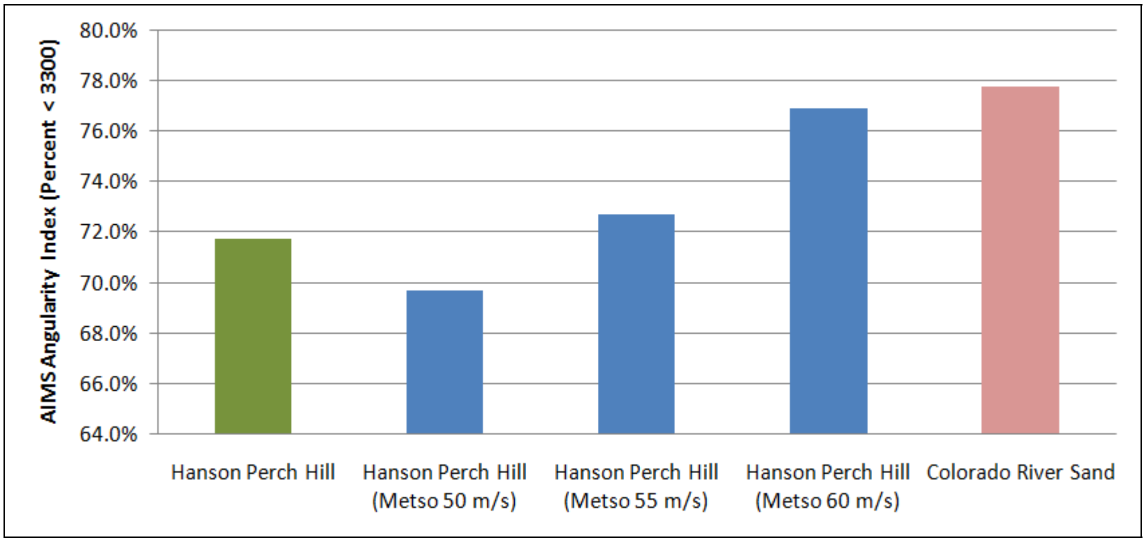

Figure 6.8: Cumulative Angularity Index for the Hanson Perch Hill Aggregate

Figure 6.9 shows the cumulative percentage of fine aggregates having an angularity factor less than 3300 for the Lattimore Stringtown aggregates. The aggregates produced by Metso were less angular than the original Lattimore aggregate, but increasing the crusher speed did not improve the angularity like it did for the Perch Hill aggregate.

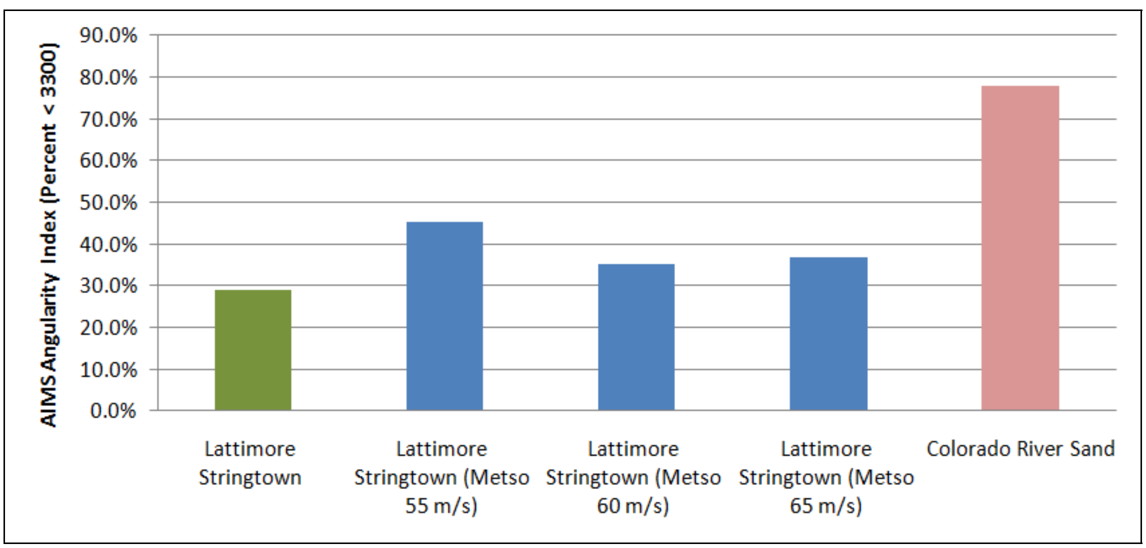

Figure 6.9: Cumulative Angularity Index for the Lattimore Stringtown Aggregate

6.3 Mortar Flow Test

In this section the effect of shape and texture of aggregates on workability was evaluated using ASTM C 1437 "Standard Test Method for Flow of Hydraulic Cement Mortar". Each of the nine sands was tested using the same mixture proportions at three different water-to-cement ratios. The mixture design for the mortars was based on a 5.5-sack concrete mixture with a sand-to-aggregate ratio of 0.37 (S/A=0.37). The volumetric proportions for the concrete mixtures are shown in Table 6.1. The volumetric proportions for the mortar mixtures in Table 6.2 were computed using the concrete proportions shown in Table 6.1

	Material Volume (%)			
w/c	**Cement**	**Water**	**Fine Aggregate**	**Coarse Aggregate**
0.39	9.74	11.97	28.97	49.32
0.405	9.74	12.43	28.80	49.03
0.42	9.74	12.89	28.63	48.74

Table 6.1: Concrete Mixture Proportions Used for the Mortar Testing

	Material Volume (%)		
w/c	**Cement**	**Water**	**Sand**
0.39	19.22	23.62	57.16
0.405	19.11	24.38	56.50
0.42	19.01	25.14	55.85

Table 6.2: Volumetric Proportions for the Mortar Mixture

A Hobart mixer was used to mix the mortar tested for flow; the mortar was prepared as follows:

1. All fine aggregates were batched oven dry.
2. Fine aggregates and water were first added to the bowl and mixed for 30 seconds at low speed.
3. The material was allowed to rest undisturbed for 30 seconds.

4. The mixer was turned on low speed, and the cement was added over a period of 30 seconds.
5. The mixer was then turned to medium speed for an additional 30 seconds and then turned off.
6. The mortar was allowed to rest for 1 minute before it was tested on the flow table.
7. The procedures described in ASTM C 1437 were used to measure the percent flow.

Only one mixture per test was performed, and all tests were performed by the same operator (the single-operator standard deviation was found to be 4% by ASTM C 1437). Results of the mortar flow tests for Perch Hill aggregates are shown in Figure 6.10. The difference in percent flow between the mixtures made with the different sands was not significant. The highest flow among the Perch Hill sands crushed by Metso was achieved by the aggregate crushed at 60 m/s. The difference in percent flow between the mixtures made with the different sands was not significant.

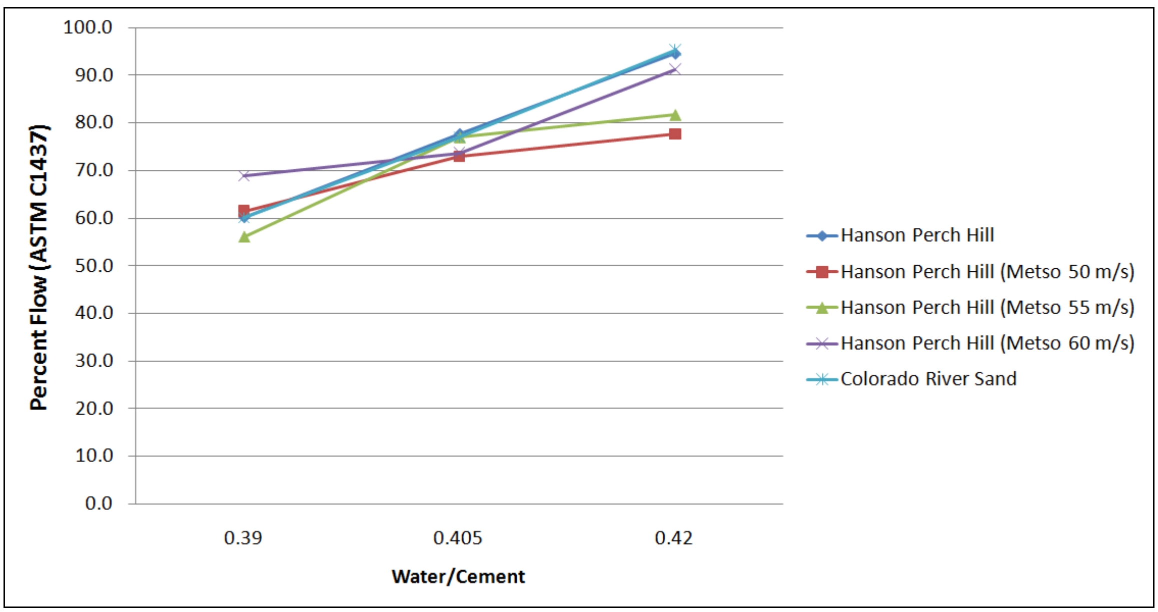

Figure 6.10: Mortar Flow Test Results for Hanson Perch Hill (Not Re-graded)

Results of the mortar flow tests for Lattimore Stringtown aggregates are shown in Figure 6.11. Among all the Lattimore Stringtown aggregates, the aggregate crushed by Metso at 65 m/s produced the mortar with the highest flow. All mortar mixtures made with aggregates crushed by Metso had higher flow than the original aggregate produced by Lattimore Materials.

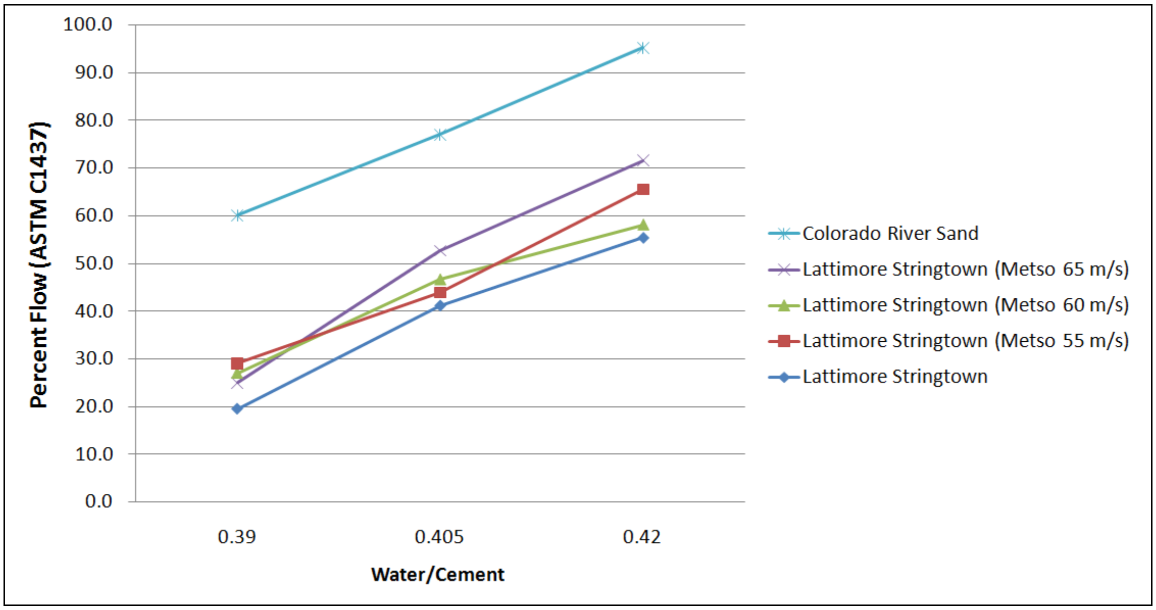

Figure 6.11: Mortar Flow Test Results for Lattimore Stringtown (Not Re-graded)

The results presented in Figures 6.10 and 6.11 are for mortar mixtures made with sands that were not re-graded; this means that sands tested were the as-received sands which had different gradations. To compare the aggregate shape and texture on the flow of mortar without having the difference in gradation affect the results, the mortar test was done on re-graded sands. All sands used were washed over a No. 200 sieve, then sieved and graded to meet the gradation shown in Table 6.3.

	% passing	% retained
#4	100	0
#8	77	23
#16	54	23
#30	30	24
#50	14	16
#100	0	14
#200	0	0

Table 6.3: Re-graded Gradation for Mortar Mixtures

The mortar proportions shown in Table 6.2 and the procedures for mixing mortar described earlier were also used. Note that the reason the grading shown in Table 6.3 was used, was because the Metso aggregates were found to have lower percentages of aggregates retained on the No. 50 and No. 100. The grading was not chosen to meet ASTM C 33 requirements since the goal of this testing was not evaluate gradations but to evaluate the effect of shape and texture on the flow of mortar.

Results for the flow of mortars made with the Perch Hill fine aggregates are shown in Figure 6.12. Even when the aggregates were re-graded, the flow of the mortar made with the aggregates crushed by Metso was not improved compared to the aggregate crushed by Hanson. The flow of the mortar made with Perch Hill sands was as high as the flow obtained by the Colorado River Sand. This does not mean that the Hanson Perch Hill Sand can be used to produce a mortar or concrete with a workability that matches the workability of a mortar or concrete made with siliceous sand. Higher flow values probably were achieved with Hanson Servtex because the absorption values used to compute the batch weights were not very accurate. This occurred because the method used to test absorption of fine aggregates (ASTM C 128) is not very repeatable and is influenced by the microfine content of an aggregate (Perch Hill has around 7% microfines).

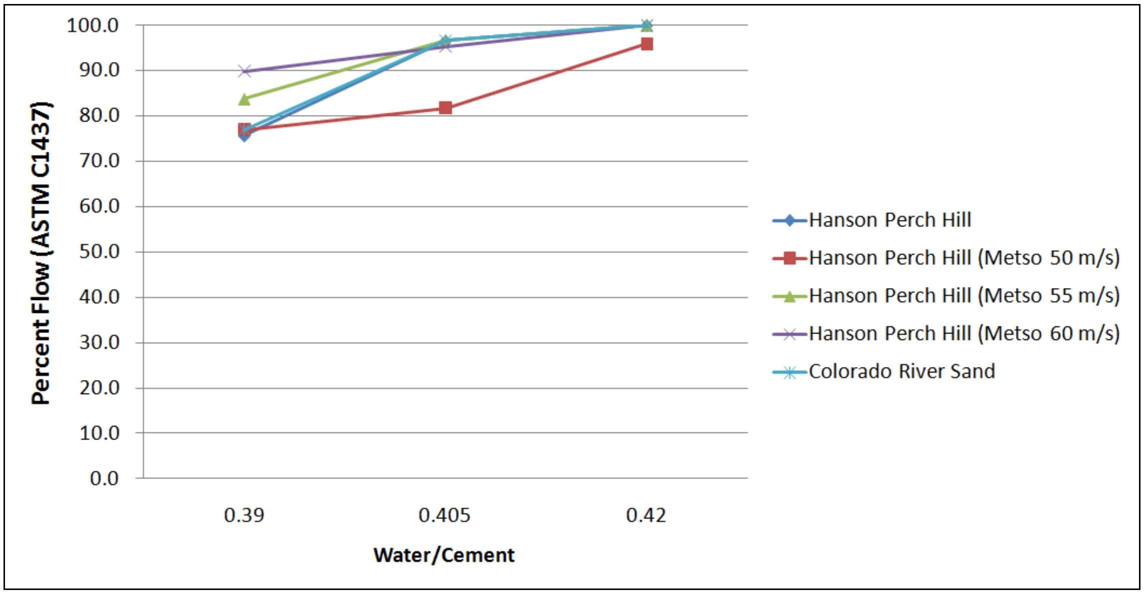

Figure 6.12: Mortar Flow Test Results for Hanson Perch Hill (Re-graded)

Figure 6.13 shows the results for the flow of mortars made with Lattimore Stringtown aggregates. The flow of the mortars made with the aggregates crushed by Metso was significantly higher than the original aggregate produced by Lattimore Materials. The highest flow was achieved by the mixture made with the aggregate that was crushed at 65 m/s. At w/c=0.39, the flow of mortar made with Lattimore Stringtown increased from around 34% to 61% when the aggregates were crushed by Metso at a speed of 65 m/s.

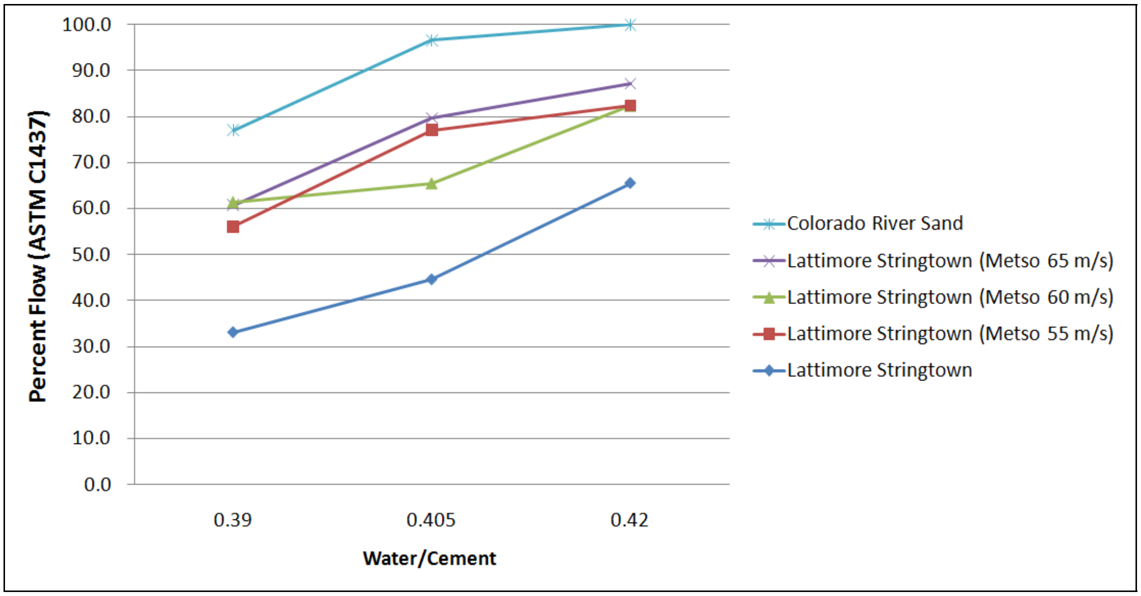

Figure 6.13: Mortar Flow Test Results for Lattimore Stringtown (Re-graded)

6.4 Conclusions

The testing done in this chapter evaluated the shape of aggregates obtained from different sources and crushed at varying speeds. Results from the three different testing methods used to evaluate the aggregate also permitted the evaluation of the test methods themselves.

MRTC was able to improve the shape of the Lattimore Stringtown aggregate using the Barmac B3000 VSI crusher. The improvement in shape was identified visually (Figure 6.3) and by testing the flow of a mortar made with the different Lattimore Stringtown aggregates (Figure 6.13). AIMS was not effective in evaluating the Lattimore Stringtown aggregates, mainly because AIMS is only capable of evaluating the 2D form and the angularity index of fine aggregates. AIMS failed to measure the flatness of the Lattimore Stringtown aggregate produced by Lattimore (the flatness of this aggregate can be visually identified in Figure 6.3). The reason AIMS is not capable of differentiating between flat and cubical aggregates is because it only evaluate the 2D form of fine aggregates. Flat aggregates set on the AIMS tray will tend to lay on their flat side, thus AIMS cannot measure the flatness of those aggregates because only the image on the non-flat side is recorded and evaluated.

The mortar flow test method used in this chapter was probably the best method used to indirectly evaluate the shape and texture of fine aggregate. The problem with the mortar flow test is that it is more time intensive (especially if re-graded sands are used). The results of the mortar flow test were also influenced by the accuracy of the measured absorption capacity of the aggregates being compared.

Chapter 7: Proportioning PCC Containing Manufactured fine Aggregates

One of the main issues encountered when manufactured fine aggregates (MFA) are used in PCC involves workability. The proportioning methods commonly used for siliceous sands do not work well for manufactured sands because MFAs have poor shape and grading (grading that does not meet ASMT C 33). This chapter describes work done to evaluate the proportioning method developed by ICAR. The only concrete property tested in this chapter was workability (slump). No hardened properties were evaluated because the hardened properties were tested using standard mixtures (Chapter 10). Optimizing mixtures by reducing cement content improves the durability of concrete without reducing strength [Fowler et al., 2008]; this is also why testing hardened property for optimized mixtures is not necessary. Suggested modifications to the ICAR method for proportioning pavement concrete are also presented at the end of this chapter.

7.1 THE ICAR PROPORTIONING METHOD

Much of the work done by ICAR involved finding better methods of proportioning MFA in concrete. The problem with using conventional methods for proportioning MFA in concrete is that those methods result in mixtures with higher cement content (or paste – water + cement). Higher cement content is not desirable because it adds to the cost of concrete and negatively affects the durability of the concrete (shrinkage, ASR, etc…). The proportioning method developed by Koehler and Fowler (2007) that was modified by McLeroy (2009) for pavement concrete was discussed in 3.4.2. This method could be summarized in the following five steps:

1. Evaluating aggregate properties.

2. Plotting the modified 0.45 power curve to determine the optimum gradation. This step involves choosing the percent of each coarse and fine aggregate needed to obtain the maximum aggregate density. The modified 0.45 power curve does not account for microfines (aggregates passing the No. 200 sieve).
3. Performing a combined dry-rodded unit weight (DRUW) test on the selected proportions of aggregates to determine the minimum paste requirements ($V_{paste\text{–}Voids}$).
4. Adding paste to achieve the desired workability ($V_{paste\_spacing}$). This paste value is based on aggregate shape. This value is also dependent on the desired workability. McLeroy (2009) determined that an addition of 3 to 8 percent paste by volume was needed for pavement concrete made with MFA containing high microfine content. To compute this value, McLeroy (2009) used a visual shape and angularity rating (Figure 7.1).

$$V_{paste\_spacing} = 3 + \left(\frac{8-3}{4}\right) \times (R_{S-A} - 1) \qquad \text{(eq. 7.1)}$$

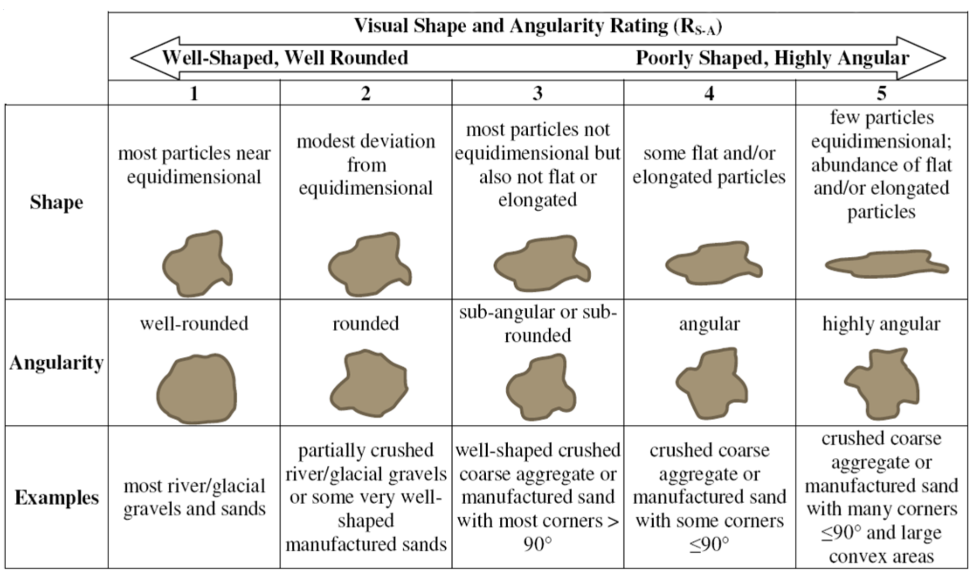

Figure 7.1: Visual Shape and Angularity Rating Scale McLeroy (2009)

5. Reducing the cement and water content based on the percentage of No. 200 fines present in the aggregates. In this step the percent microfines in the aggregates are subtracted from the paste content (paste = cementitious materials + water) while maintaining a constant water-to-cementitious ratio. Microfines are not accounted for as cementitious materials but as powder (powder = cementitious + microfines).

7.2 PRELIMINARY MODIFICATIONS TO THE ICAR PROPORTIONING METHOD

Before any testing was done to evaluate the ICAR method, the fourth step which involved determining the additional paste needed based on the shape of the aggregates was modified. The modification was made because the visual shape and angularity rating used by McLeroy (2009) was subjective. Instead of using Figure 7.1, the 2D form and angularity indices determined using AIMS were used. As discussed in Chapter 6, AIMS evaluates the shape of fine aggregates by using a 2D form index. The lower the form index the more equidimensional a particle is. AIMS also evaluates the angularity of fine aggregates. The scale used ranges from 0 to 10000; 0 indicates the presence of well round aggregates, and 10000 indicates the presence of highly angular aggregates. Note that this work was done before the results of Chapter 6 were obtained. Those results showed that AIMS was not capable of properly evaluating flat fine aggregates.

In the third step of the ICAR method the minimum paste is computed using the dry-rodded unit weight. This minimum paste is the paste required to coat the aggregates. The additional paste, which is computed in the fourth step of the ICAR method, is the paste needed to achieve the desired workability. The percentage of that paste is related to its flow; the higher the flow, the less of that additional paste needed to achieve the required workability. For this reason, a flow test was run on four aggregates and the results were compared to the AIMS values to determine how flow and AIMS values relate. The tested aggregates included the Colorado River Sand, Hanson Servtex, Texas Crushed Stone, and Capital Marble Falls (those aggregates were not as flat as Lattimore Stringtown was). The aggregates were designated with numbers based on their shape; aggregate 1 was a natural sand with a good shape, Aggregate 2 was a manufactured fine aggregate (MFA) with a good shape, Aggregate 3 and 4 were MFA with relatively poor shape. For each of those aggregates the cumulative percentage of aggregate with a shape ≤ 6 was computed (Table 7.1 – highlighted in yellow). Aggregate 1 had the highest percentage (indicating a good shape factor), Aggregates 3 and 4 had the worse shape factor (lower cumulative percentages).

	Aggregate 1 (Cum. %)	Aggregate 2 (Cum. %)	Aggregate 3 (Cum. %)	Aggregate 4 (Cum. %)
≤ 6	42.7	29.8	17.8	14.7
≤ 12	96.6	95.1	93.5	93.8
≤ 20	100	100	100	100

Table 7.1: Cumulative 2D Form Index

Table 7.2 shows the results of the cumulative angularity index for the four aggregates used in this study. Aggregates 1 and 2 had higher percentage of particles with an angularity index ≤ 3300; therefore they were less angular than aggregates 3 and 4.

	Aggregate 1 (Cum. %)	**Aggregate 2 (Cum. %)**	**Aggregate 3 (Cum. %)**	**Aggregate 4 (Cum. %)**
≤ 3300	79.1	68.2	50.4	55.6
≤ 6600	99.7	99.2	98.5	100
≤ 10000	100	100	100	100

Table 7.2: Cumulative Angularity Index

Note that Aggregate 1 had the best shape and angularity index while aggregate 2 had a shape and angularity index that was better than aggregates 3 and 4. Aggregate 3 had a better shape index than aggregate 4 but had a lower angularity index.

To evaluate whether or not those indices of shape and angularity could relate to concrete workability, the flow of mortars made with these aggregates was evaluated. ASTM C 1437 was used to measure the flow of mortar. To evaluate shape and angularity without including the effect of gradation, all fine aggregates were washed, sieved, and then re-graded to have the same gradation. All mortar mixtures had the same volume (1 liter) and were batched based on SSD values with no additions of admixtures. The same procedures described in section 6.3 were used. The results obtained are shown in Figures 7.2 and 7.4.

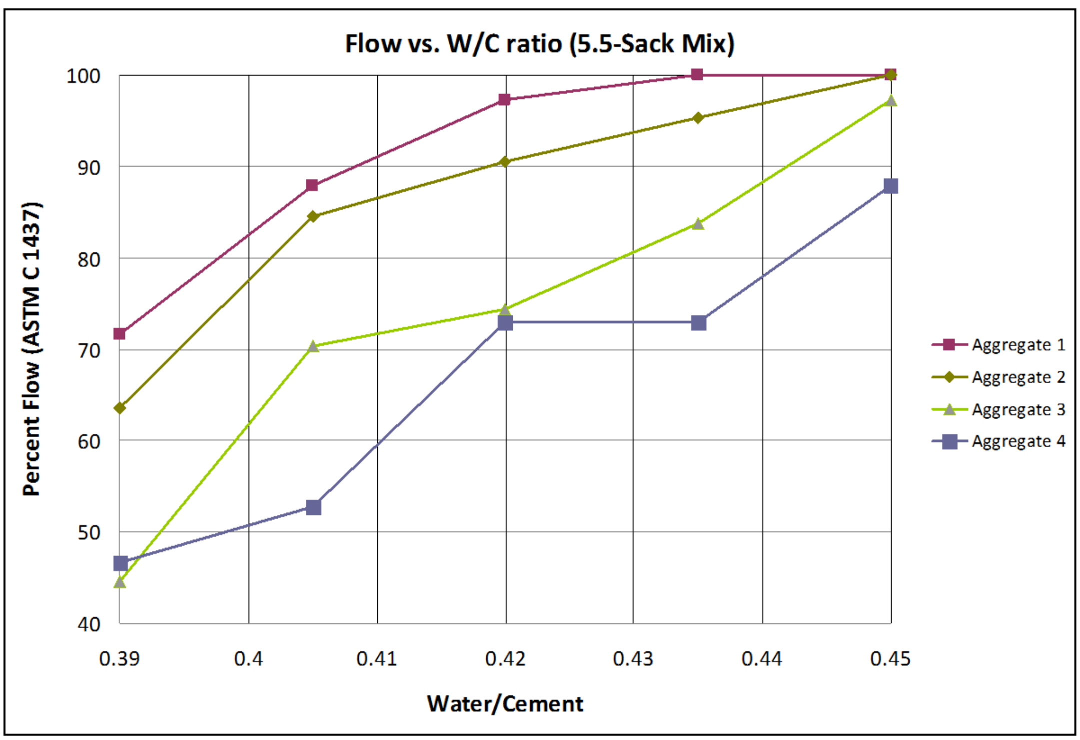

Figure 7.2: Flow of aggregates with different shape and angularity (5.5 sacks)

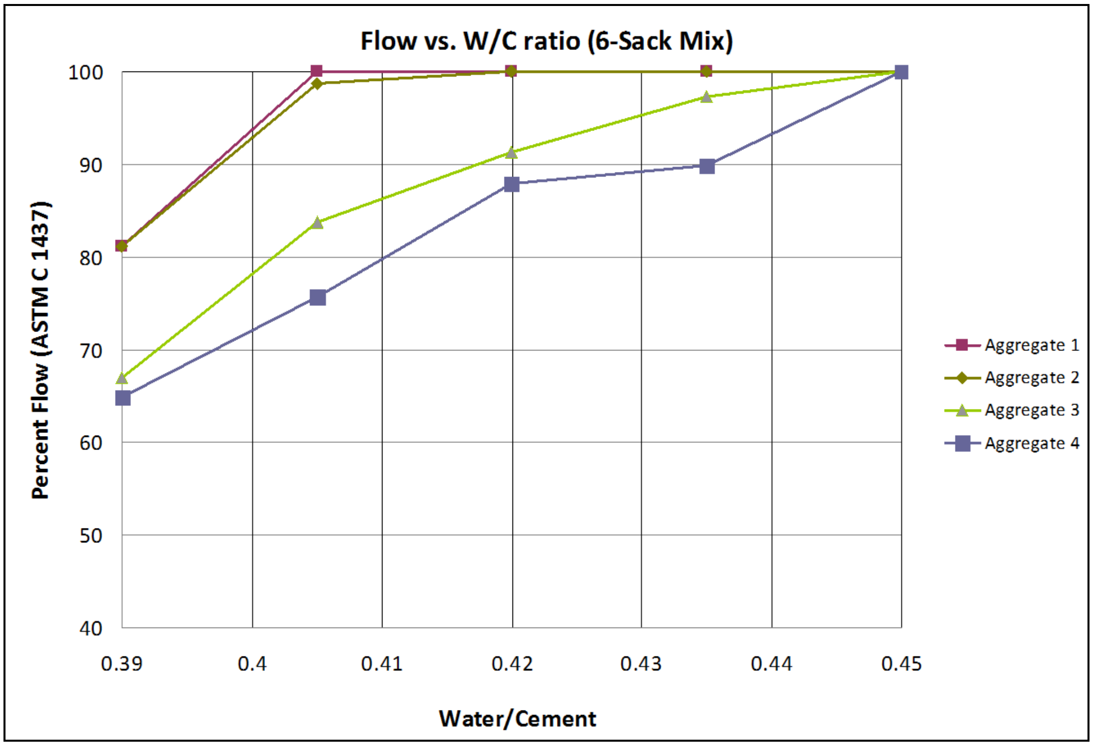

Figure 7.3: Flow of aggregates with different shape and angularity (6 sacks)

Results in Figure 7.2 represent a concrete mortar composed of a 5.5-sack mix, while Figure 7.3 represents a concrete mortar of a 6-sack mix. Comparing Figures 7.2 and 7.3 to results of Tables 7.1 and 7.2, shows that aggregates with higher 2D form and angularity index performed better than aggregates with lower indices. Thus shape and angularity values obtained from AIMS seemed to relate to concrete flow measured using ASTM C 1437.

The shape of aggregates measured by AIMS seemed to influence flow more than angularity did (comparing aggregate 3 and 4). Based on those results, the visual rating chart was replaced by a linear AIMS function that increased the additional paste content in the mixture (up to 8%) as the AIMS form and angularity values increased. The function was made such that an increase in form would account for 80% of the increase in paste while the other 20% was associated to the angularity index. The goal was to start out with a basic AIMS function and to then to improve this function based on the slump results obtained from testing fine aggregates with various shapes. Note that the AIMS function was only used to evaluate fine aggregate shape since the shape of fine aggregates affect concrete workability more than the shape of coarse aggregates. The ICAR method also accounts for the shape of coarse aggregates indirectly through the combined dry-rodded unit weight test in step three.

7.3 EVALUATING THE ICAR METHOD

The ICAR method was evaluated using four different fine aggregates and one coarse aggregate; these included the Colorado River Sand, Capital Marble Falls (MFA), Hanson Servtex (MFA), Texas Crushed Stone (MFA), and the Perch Hill coarse aggregate (Grade 4). All aggregate properties were first evaluated (Chapter 4) before the proportioning computations were performed (step one of section 7.1). One mixture was batched and tested for each mixture proportion considered in this chapter.

The second step of the ICAR method involves determining the optimum aggregate proportions using a modified 0.45 chart (discussed in 2.5.3); this was done for the four fine aggregates (Figures 7.4. 7.6, 7.8 and 7.9). The optimum grading using the modified 0.45 power chart was found to be at a sand-to-aggregate ratio of 0.3. This gradation was judged to be too coarse, so a different sand-to-aggregate ratio was also considered for two of the four fine aggregates. The second sand-to-aggregate ratio was determined using the conventional 0.45 power chart; the optimum gradation using such a curve corresponded to a sand-to-aggregate ratio of around 0.37 (Figures 7.5 and 7.7).

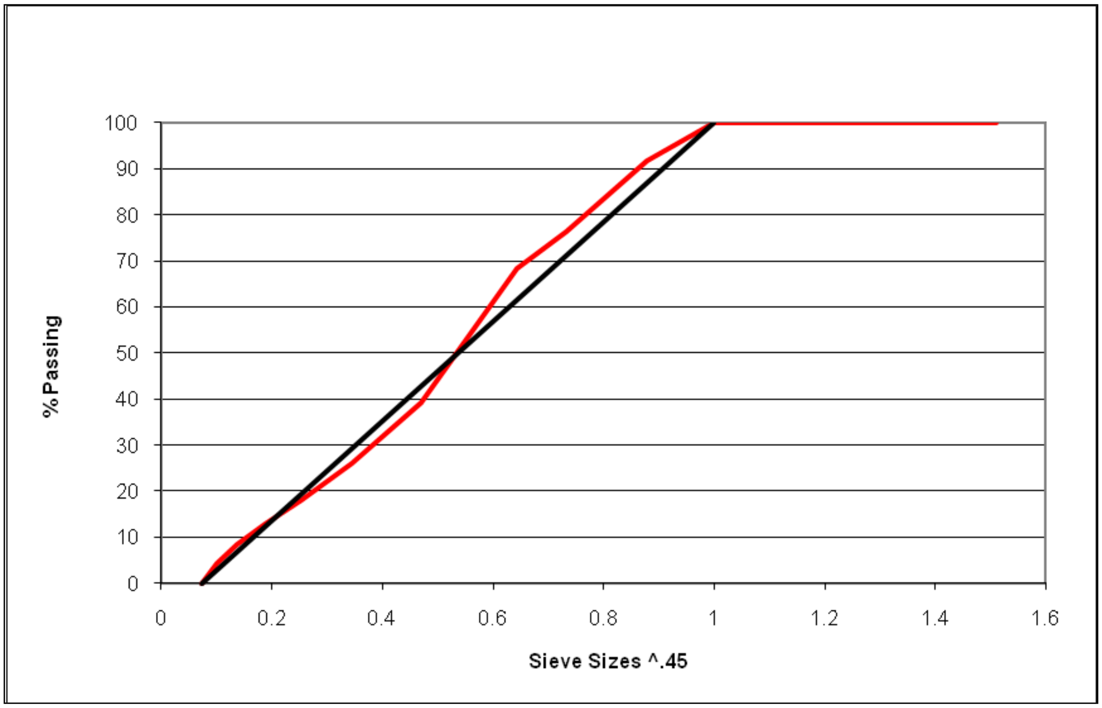

Figure 7.4: Capital Marble Falls S/A=0.30 (Modified 0.45 Power Chart)

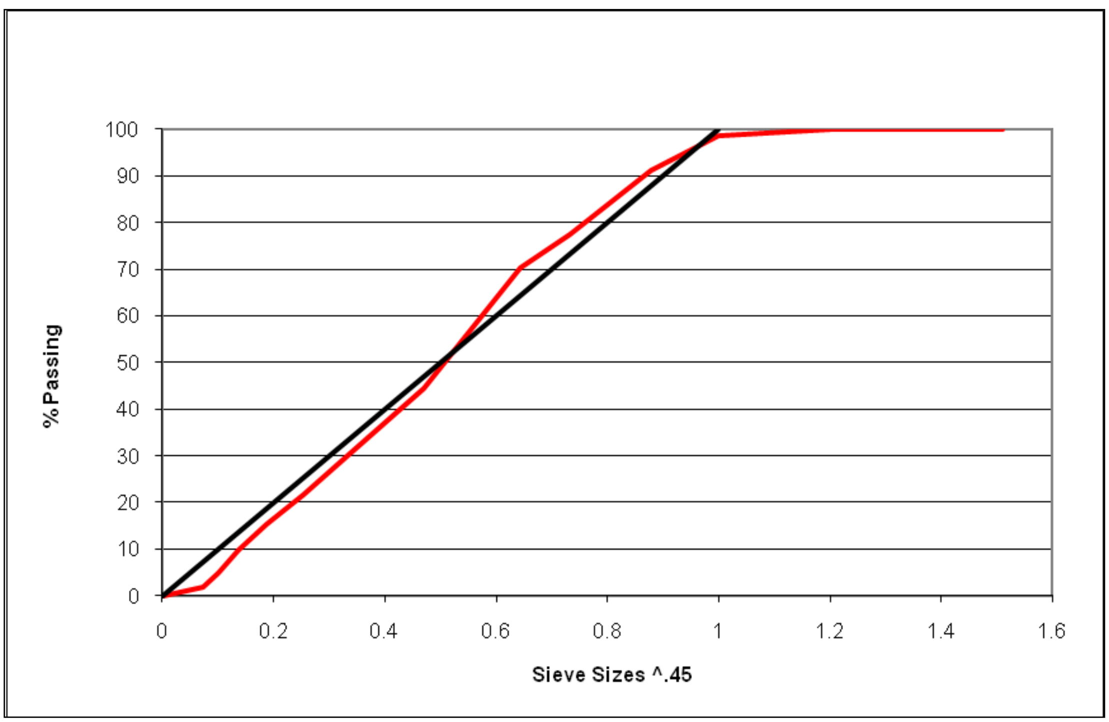

Figure 7.5: Capital Marble Falls S/A=0.37 (Conventional 0.45 Power Chart)

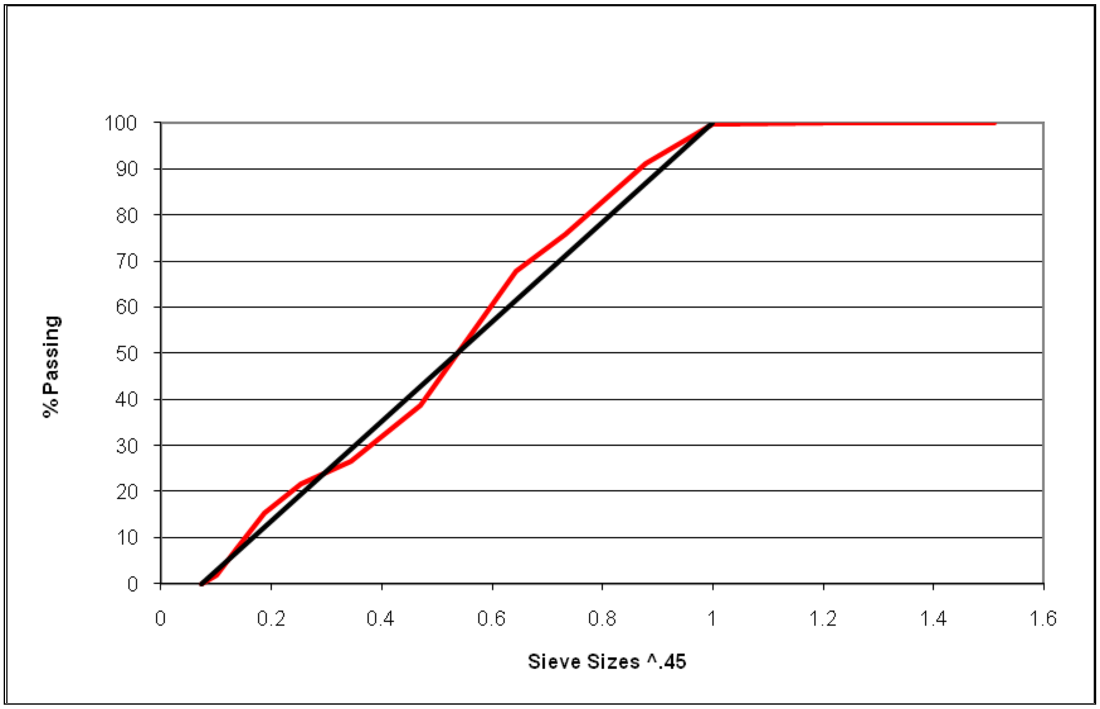

Figure 7.6: Colorado River Sand S/A=0.30 (Modified 0.45 Power Chart)

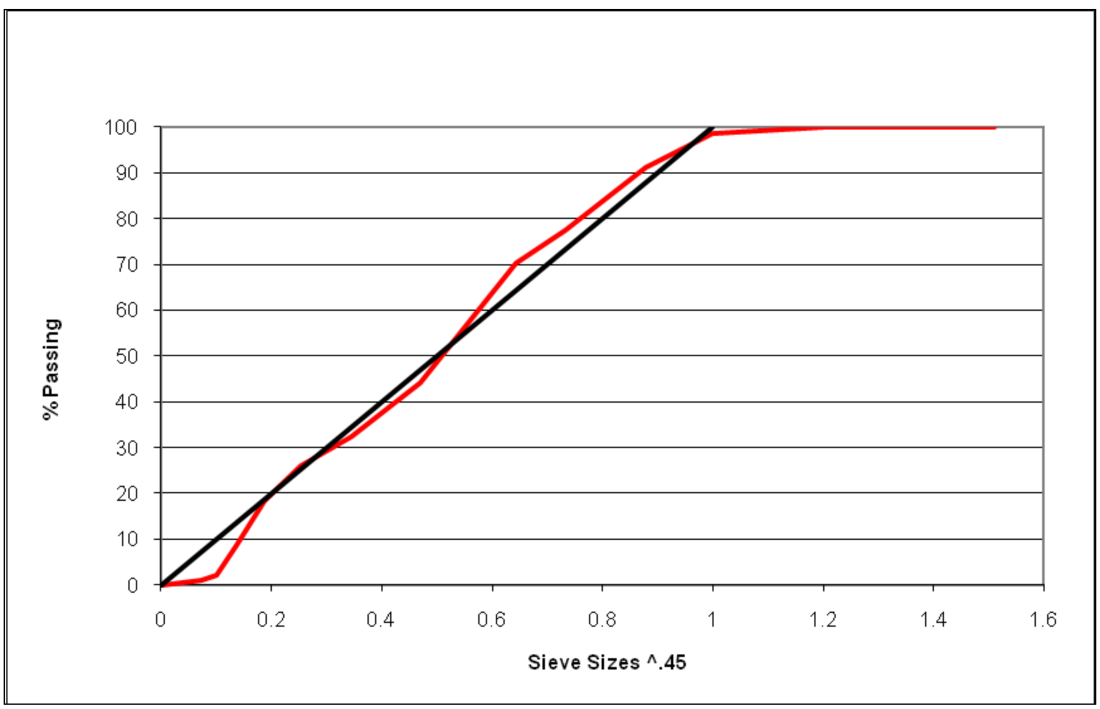

Figure 7.7: Colorado River Sand S/A=0.37 (Conventional 0.45 Power Chart)

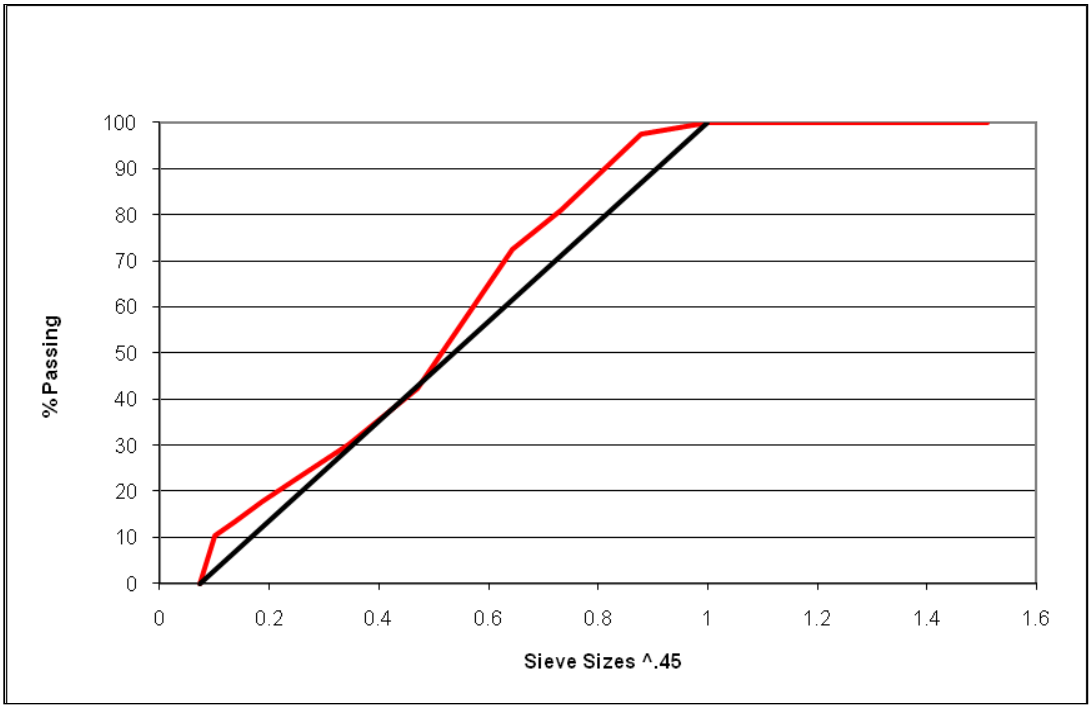

Figure 7.8: Texas Crushed Stone S/A=0.30 (Modified 0.45 Power Chart)

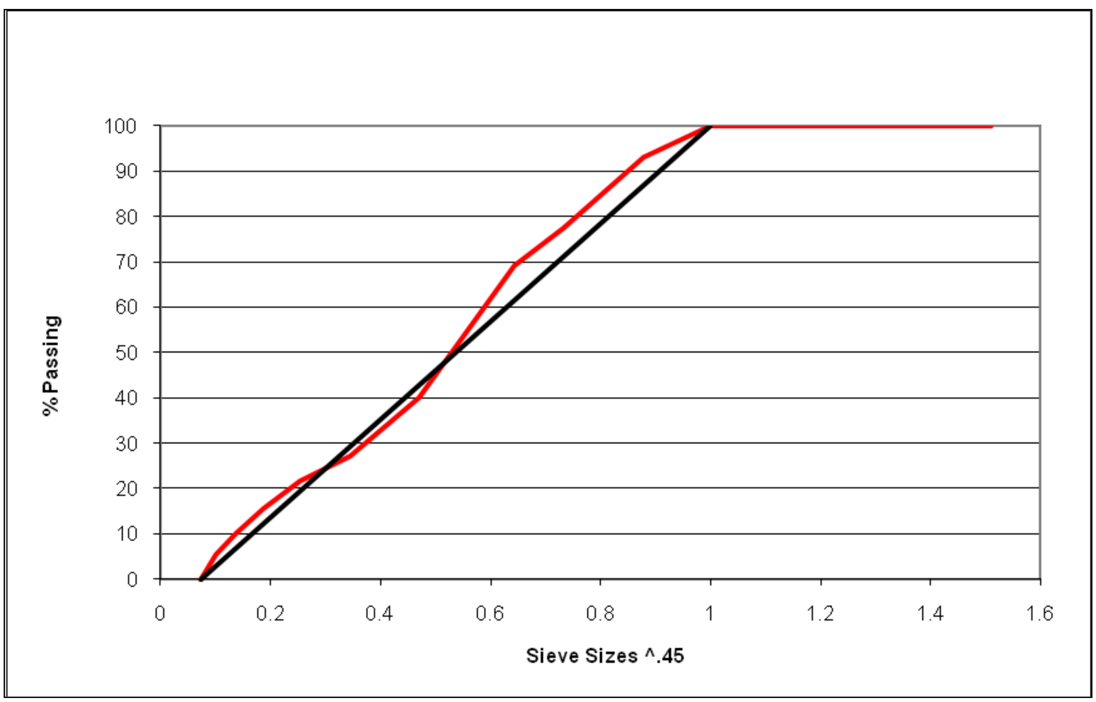

Figure 7.9: Hanson Servtex S/A=0.30 (Modified 0.45 Power Chart)

The third step in the ICAR method is to measure the combined dry-rodded unit weight (DRUW) of the aggregates. DRUW was measured for the six aggregate combinations shown in Figures 7.4 through 7.9. For the Colorado River Sand and the Capital Marble Falls MFA, a higher DRUW was obtained at a sand-to-aggregate ratio of 0.3 (higher DRUW indicates a higher aggregate density). After DRUW was determined, the minimum paste content was computed. The fourth and fifth step involved adding paste based on the shape of the aggregate and then reducing the paste volume based on the microfine content of the sand. The results obtained are shown in Table 7.3. Each of the proportions shown in Table 7.3 was batched oven-dry and then mixed following the procedures described in ASTM C 192. The minimum dosage of WRDA 82 was added to each of those mixtures (2 oz/cwt). The slump test was then performed as described by ASTM C 143. All mixtures made with sand-to-aggregate ratio of 0.3 resulted in a shear slump. A shear slump occurs when the top portion of the concrete shears off during a slump test; it is usually due to the mixture being too coarse or due to a lack of paste. In this case it was clear that the shear slump was a result of having mixtures that were too coarse. The mixtures made with Capital Marble Falls and the Colorado River Sand at a sand-to-aggregate ratio of 0.37 achieved a slump that was higher than required (½ to 2 ½ -in. is required for slip-form paving concrete). The reason this occurred was because too much cement was used. For Capital Marble Falls MFA, using the ICAR proportioning method resulted in 7 ½ sack mixture. The reason such high cement content was computed using this method is because step four adds a certain paste quantity that is based on the shape of the fine aggregate, and then step five subtracts a portion of the paste based on the microfine content. Capital Marble Falls is a fine aggregate with poor shape but with very low microfine content, so step four resulted in the addition of paste, but no paste was subtracted in step five because the aggregate did not contain many microfines.

	Capital Marble Falls (MFA)		Colorado River Sand (Natural)		Texas Crushed Stone (MFA)	Hanson Servtex (MFA)
Sand/Aggregate	0.37	0.30	0.37	0.30	0.30	0.30
W/C	0.45	0.45	0.45	0.45	0.45	0.45
Coarse Aggregates (lb/yd^3)	1861.8	2107.3	2006.6	2274.2	2274.7	2214.0
Fine Aggregates (lb/yd^3)	1138.5	940.6	1147.6	949.1	938.6	913.3
Water (lb/yd^3)	319.1	307.0	268.65	254.6	254.5	273.5
Cement (lb/yd^3)	709.2	682.2	597.01	565.9	565.6	607.9
Sacks of Cement (lb/yd^3)	7.54	7.26	6.35	6.02	6.02	6.47
Slump (in.)	7- ¾	Shear Slump	3- ¾	Shear Slump	Shear Slump	Shear Slump

Table 7.3: Summary of the Results Obtained Using the ICAR Proportioning Method

7.4 Determining the Optimum Paste Content for Pavement Concrete

The ICAR method developed by McLeroy (2009) worked for MFA containing high microfines but it did not work well with MFA not containing high microfine content. The problem encountered occurred after the minimum paste content was computed in step three. For this reason, a series of tests was made to determine the minimum paste addition needed to achieve the desired workability. Examples are shown in Tables 7.4 and 7.5.

Table 7.4 and 7.5 show that the slump requirement was reached by just using the minimum paste volume computed using DRUW. In Table 6.2, a slump of 4 in. was obtained for the mixture containing no additional paste; a lower slump could be obtained if a lower dosage of admixture was used. The minimum paste content determined for the mixtures containing Capital Marble Falls corresponded to a 6.2-sack mixture. To obtain mixtures with lower cement content, a better gradation of coarse aggregate with a larger maximum size aggregate is needed. The coarse aggregate used for this testing was a grade 4; pavement mixtures are usually made with a grade 3 coarse aggregate or a combination of a grade 2 and grade 4. Compared to grade 4, both grades 2 and 3 have larger maximum size aggregates.

Sacks of Cement	Paste Added	Cement (lb/yd^3)	Water (lb/yd^3)	Fine Aggregate (lb/yd^3)	Coarse Aggregate (lb/yd^3)	Slump (in.)	WRDA Dosage (oz/cwt)
6.2	0	581.9	261.9	1238.9	2026.1	4.0	9.4
6.4	1	603.9	271.7	1221.6	1997.7	2.0	6.0
6.7	2	625.8	281.6	1204.3	1969.4	3- ½	5.3
6.9	3	647.8	291.5	1187.0	1941.1		
7.1	4	669.7	301.4	1169.6	1912.7	7- ½	5.5

Table 7.4: Determining the Optimum Paste Content for a Mixture Containing Capital Marble Falls and a w/c=0.45

Sacks of Cement	Paste Added	Cement (lb/yd^3)	Water (lb/yd^3)	Fine Aggregate (lb/yd^3)	Coarse Aggregate (lb/yd^3)	Slump (in.)	WRDA Dosage (oz/cwt)
6.4	0	605.6	254.4	1238.9	2026.1	1- ¼	7.8
6.7	1	628.4	263.9	1221.6	1997.7	1- ¾	6.7
6.9	2	651.3	273.5	1204.3	1969.4	5.0	7.0
7.2	3	674.1	283.1	1187.0	1941.1	2.0	5.4

Table 7.5: Determining the Optimum Paste Content for a Mixture Containing Capital Marble Falls and a w/c=0.42

The same procedure shown in Figures 7.4 and 7.5 was applied for five other sands at a water-to-cement ratio of 0.45 and a sand-to-aggregate ratio of 0.37. A summary of the results showing the percentage of additional paste required to achieve ½ to 2 ½ -in. of slump is shown in Table 7.6. More paste was needed for the mixture made with Texas Crushed Stone because that MFA contained about 21% microfines. Lattimore Stringtown required less paste than the computed minimum paste requirement. This occurred because the combined DRUW measured for Lattimore Stringtown was relatively low. A low DRUW resulted in a high computed minimum paste content. Lattimore Stringtown has a low packing density when measured using DRUW; after mixing it in concrete, its packing density seemed to improve.

Fine Aggregate Source	Additional Paste Needed to Reach Target Slump (%)
Colorado River Sand	0
Capital Marble Falls	0
Texas Crushed Stone	2
TXI Bridgeport	0
Hanson Perch Hill	0
Lattimore Stringtown	-3

Table 7.6: Additional Paste Required to Reach Target Workability

7.5 CONCLUSIONS AND RECOMMENDATIONS

The ICAR proportioning method proposed by McLeroy (2009) for pavement concrete overestimates that amount of cement needed for mixtures containing MFA with low microfine content. To avoid overestimating the cement content it is recommended to only compute the minimum paste content and not to add any additional paste before trial batches are evaluated (steps one to three of 7.1). Since pavement concrete is a low slump concrete, the minimum paste computed using DRUW should be enough to achieve ½ to 2 ½-in. slump (unless the MFA contains high microfines). However, if the slump is too low, the slump can be adjusted by making the paste more flowable; this can be achieved by adding a higher dosage of admixture.

The modified 0.45 power curve seemed to result in denser aggregate gradations, but it also resulted in aggregate proportions that caused shear slumps for the fine aggregates tested. The modified 0.45 power curve was developed by Fowler and Koehler (2007) for self-consolidating concrete (SCC), but the conventional 0.45 curve seemed to work better for proportioning pavement concrete.

Chapter 8: Preliminary Skid Testing

Most of the work done by the International Center for Aggregate Research (ICAR) at the University of Texas mainly involved finding better proportioning methods for MFA and testing hardened properties of concrete made with MFA including strength, modulus of elasticity, abrasion resistance, and coefficient of thermal expansion. However, none of the ICAR projects evaluated skid resistance of concrete. When this research project started in fall 2008, it was not clear what methods were going to be used to measure skid resistance in the laboratory. TxDOT, the sponsor of the project suggested using the Circular Track Meter (CTM – discussed in 3.3.1) and the Dynamic Friction Tester (DFT – discussed in 3.3.2) to evaluate skid resistance of concrete in the laboratory and in the field. Those two devices had previously been used on a similar TxDOT project that involved evaluating skid resistance of asphalt pavements. The CTM and DFT are shown in Figures 8.1 and 8.2.

Figure 8.1: Circular Track Meter (CTM)

Figure 8.2: Dynamic Friction Tester (DFT)

Most of the research that had been found on the usage of those devices was research done on asphalt concrete. For this reason, preliminary testing was done to evaluate the two devices. The goal of the preliminary testing described in this chapter was to:

1. Better understand how the texture and friction values obtained from the CTM and DFT relate to concrete textures.
2. Establish a testing protocol for evaluating the polish resistance of concrete.

8.1 EVALUATING THE TEXTURE OF CONCRETE MADE BY DIFFERENT FINISHING TECHNIQUES

Five surfaces having different textures were evaluated. Four of those were 24-in. by 24-in. concrete slabs that were cast in the laboratory. The same mixture was used to cast all four concrete surfaces. The surfaces evaluated using the CTM and DFT had:

- A broom finish (Figure 8.3)
- Burlap drag finish (Figure 8.4)
- Tined & burlap drag finish (Figure 8.5)
- A trowel finish (Figure 8.6); a steel trowel was used to obtain a smooth surface.

- The fifth surface evaluated was a smooth glass surface. The reason this surface was evaluated was to measure how low of a friction and texture value could be obtained using those devices. Since the CTM is a laser based device, the surface of the glass was painted after friction measurements using the DFT were taken.

Figure 8.3: Broom Finish

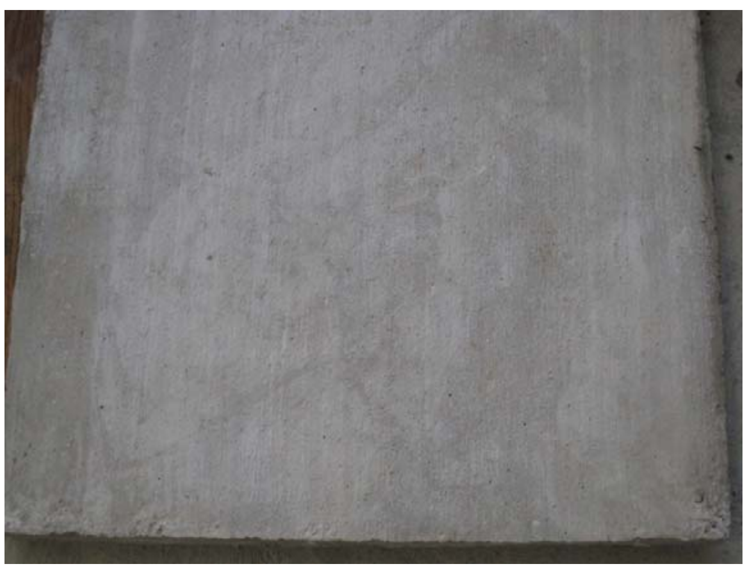

Figure 8.4: Burlap Drag

Figure 8.5: Tined + Burlap Drag

Figure 8.6: Trowel Finish

Figure 8.7: Painted Glass

The CTM was used to measure texture on three different locations on each of the surfaces. The profiles obtained for the different surfaces using the CTM are shown in Figure 8.8. The CTM measures the Mean Profile Depth (MPD); the MPD is a measure of the macro-texture of a surface. The range of the MPD values for the five surfaces is shown in Figure 8.9. The highest MPD was obtained by using a broom finish. Tining a surface significantly increased the MPD values measured. The MPD value of a surface finished with a burlap drag almost doubled after the surface was tined. The surface that was trowel finished had the lowest MPD value among the four concrete specimens. The glass surface had very low MPD values; the values were higher than zero because the surface was painted.

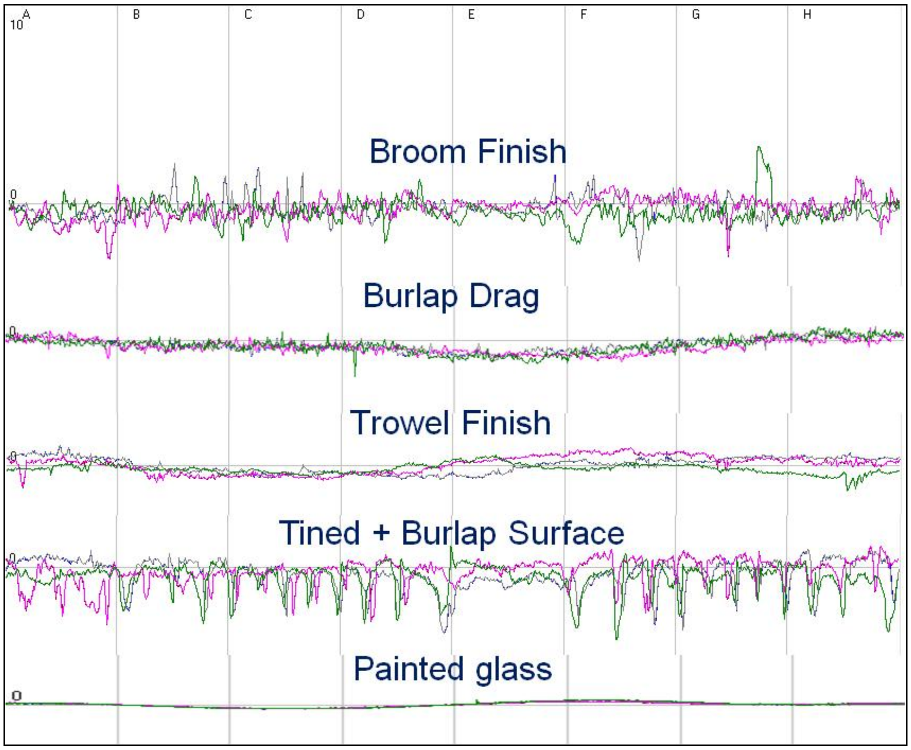

Figure 8.8: Texture Profiles

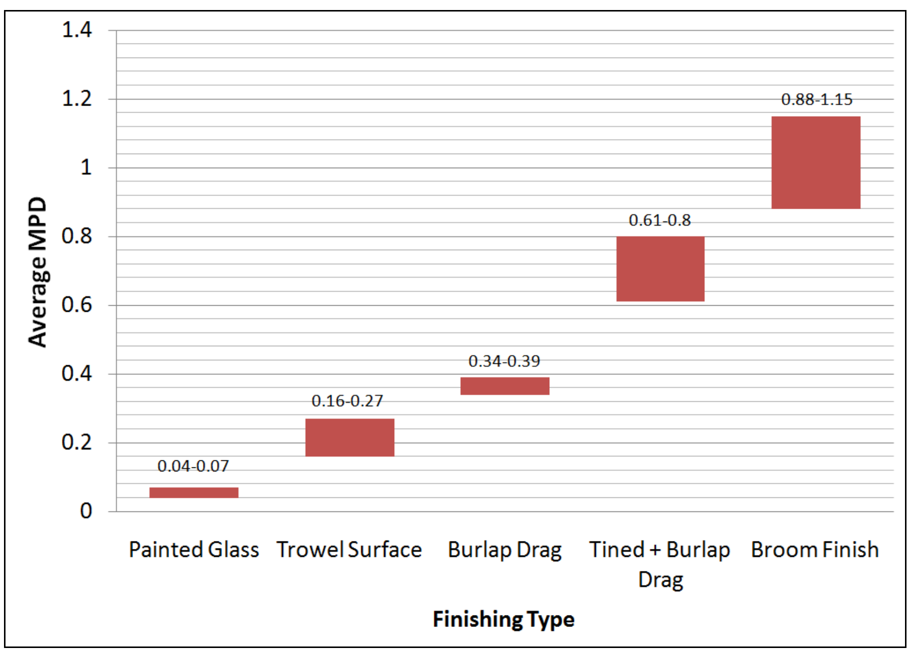

Figure 8.9: Measured MPD range for the Different Textures

The DFT was used to evaluate the friction on the five surfaces. The DFT measures the coefficient of friction on a surface from a speed of 0 to 80 km/hr. The average of three readings taken on three different locations on each surface is shown in Figure 8.10. Using the values shown in Figure 8.10, the coefficient of friction obtained at specified speeds could be obtained. The coefficient of friction at 20 km/hr and 60 km/hr are shown in Figures 8.11 and 8.12. The coefficient of friction at 20 km/hr was higher than the coefficient of friction at 60 km/hr. The values for the coefficient of friction (Figures 8.11 and 8.12) do not correlate well with the texture values (Figure 8.9). The higher textures obtained from the broom and the tined finishes did not significantly increase the coefficient of friction. Tining a surface finished with a burlap drag did not lead to a major increase in the coefficient of friction value at 60 km/hr. The only surface that had significantly lower friction was the glass surface.

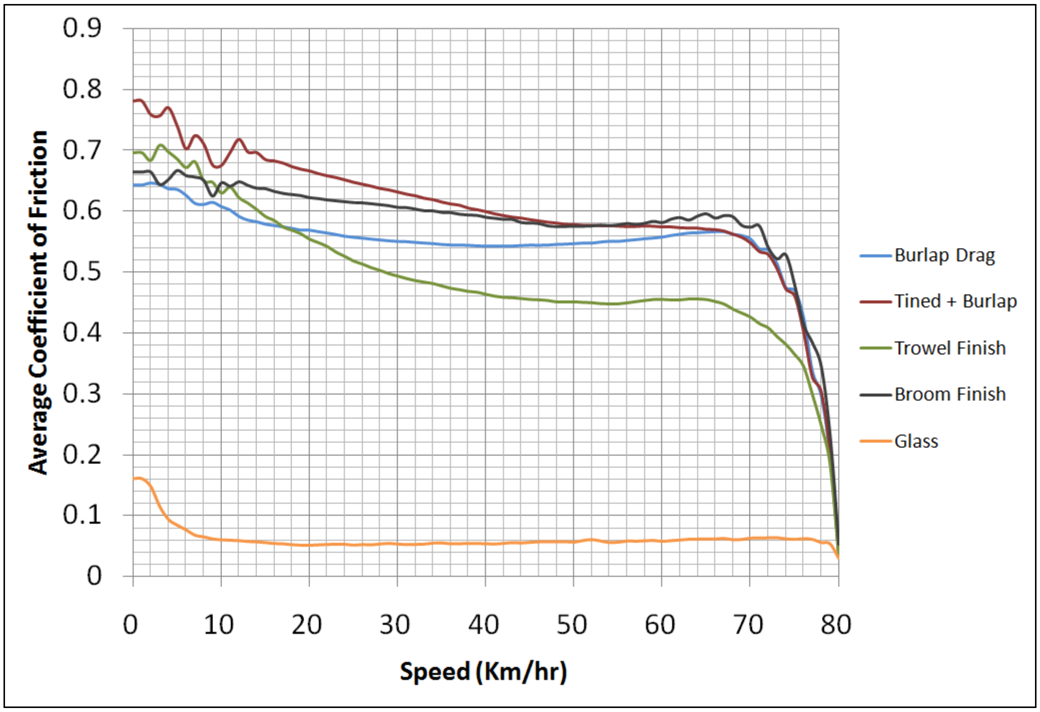

Figure 8.10: DFT Values for the Different Textures

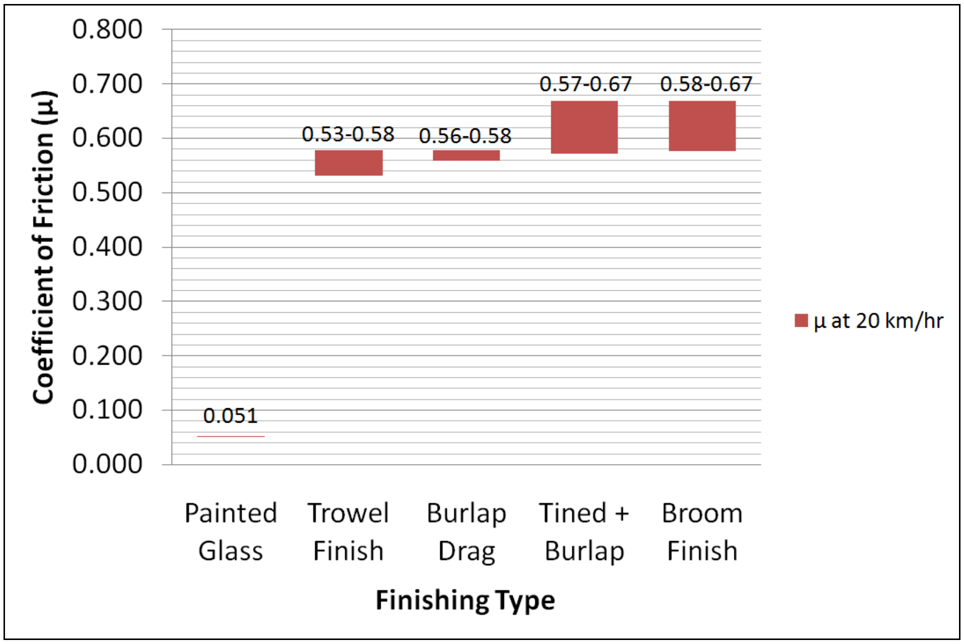

Figure 8.11: DFT20 Values for Different Textures

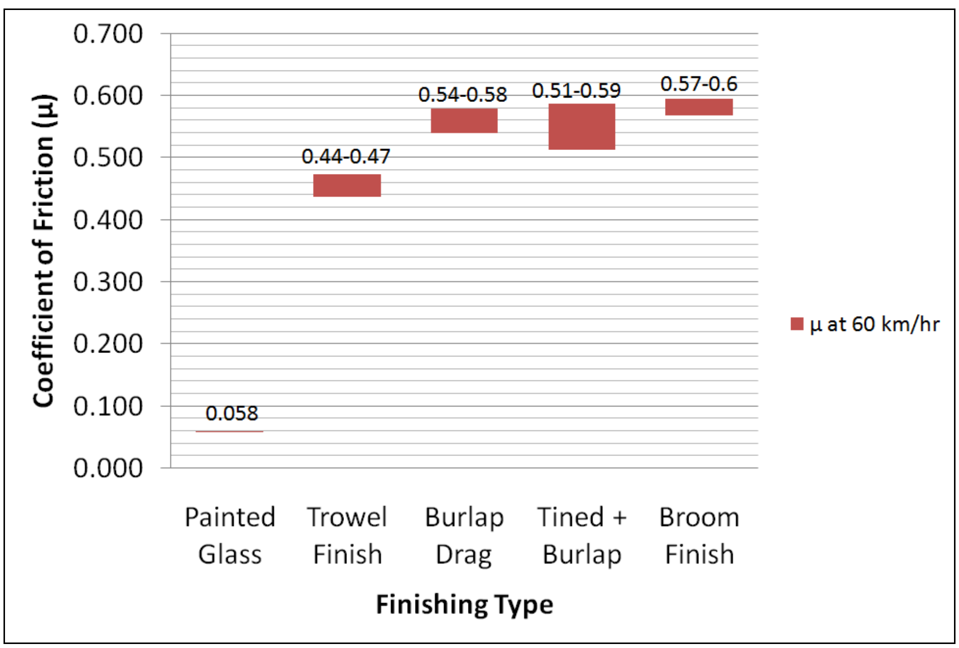

Figure 8.12: DFT60 Values for Different Textures

Results show that the CTM can differentiate between the different types of finishing techniques. The CTM is a device that is good for evaluating macro-texture; macro-texture in PCC is intentionally formed to provide skid resistance and to drain water from concrete pavements. Macro-texture formed on concrete is not expected to contribute to long-term skid resistance (reviewed in 2.1.5 and 3.2.3.2). The DFT cannot differentiate between the types of finishings created on the surface of the concrete because the DFT evaluates friction; having higher macro-texture does not seem to always lead to higher friction values and that is why no clear correlation exists between the MPD values and friction values. For example, the surface finished using a trowel had a low MPD value compared to the tined surface (Figure 8.9), but the differences in friction values (Figures 8.11 and 8.12) were not as significant. Moreover, although the surface finished with a trowel had very low MPD, it had considerably higher friction values compared to the glass surface. This further shows that the friction coefficient measured by the DFT is mainly controlled by the micro-texture and not the macro-texture.

8.2 ESTABLISHING A TESTING PROTOCOL FOR TESTING TEXTURE AND FRICTION AT THE LABORATORY

To be able to evaluate the polish resistance of concrete, a method of simulating abrasion due to traffic was needed. A Three-Wheel Polishing Device (TWPD) developed by the National Center for Asphalt Technology (NCAT) was purchased (Figure 8.13); the TWPD was developed to be used with a CTM and DFT. It polishes a circular path on a laboratory specimen that has the same diameter as the path evaluated by the CTM and DFT. NCAT developed the polisher to test asphalt concrete. The NCAT polisher is composed of three wheels that rotate on a laboratory specimen for a specified amount of cycles. Circular iron plates can be placed on the turntable to change the weight on the TWPD. The TWPD also has a water spray system that sprays water on the surface being polished. NCAT added the water spray system to wash away the abraded particles, simulate wet weather conditions, and to extend the life of the wheels because their initial testing showed that wheels were getting worn faster when no water was used. The introduction of water in the polisher is believed to cool the wheel material and reduce tire wear (wheels would need to be changed less often). NCAT investigated the use of several types of wheel material to polish asphalt surfaces and chose pneumatic rubber wheels.

Figure 8.13: NCAT Three-Wheel Polishing Device

When the TWPD was obtained, it was important to investigate whether or not the TWPD was able to abrade PCC specimen. Abrasion on PCC pavements is usually caused by trucks and not by cars, so the TWPD was loaded with the maximum amount of plates to attain the highest stress. Wheels made with three different materials were tested; the materials tested included:

- Rubber
- Polyurethane
- Steel

Several surfaces were tested to investigate which of those wheel materials could serve as a better accelerated wear test. The four different wheels tested are shown in Figure 8.14.

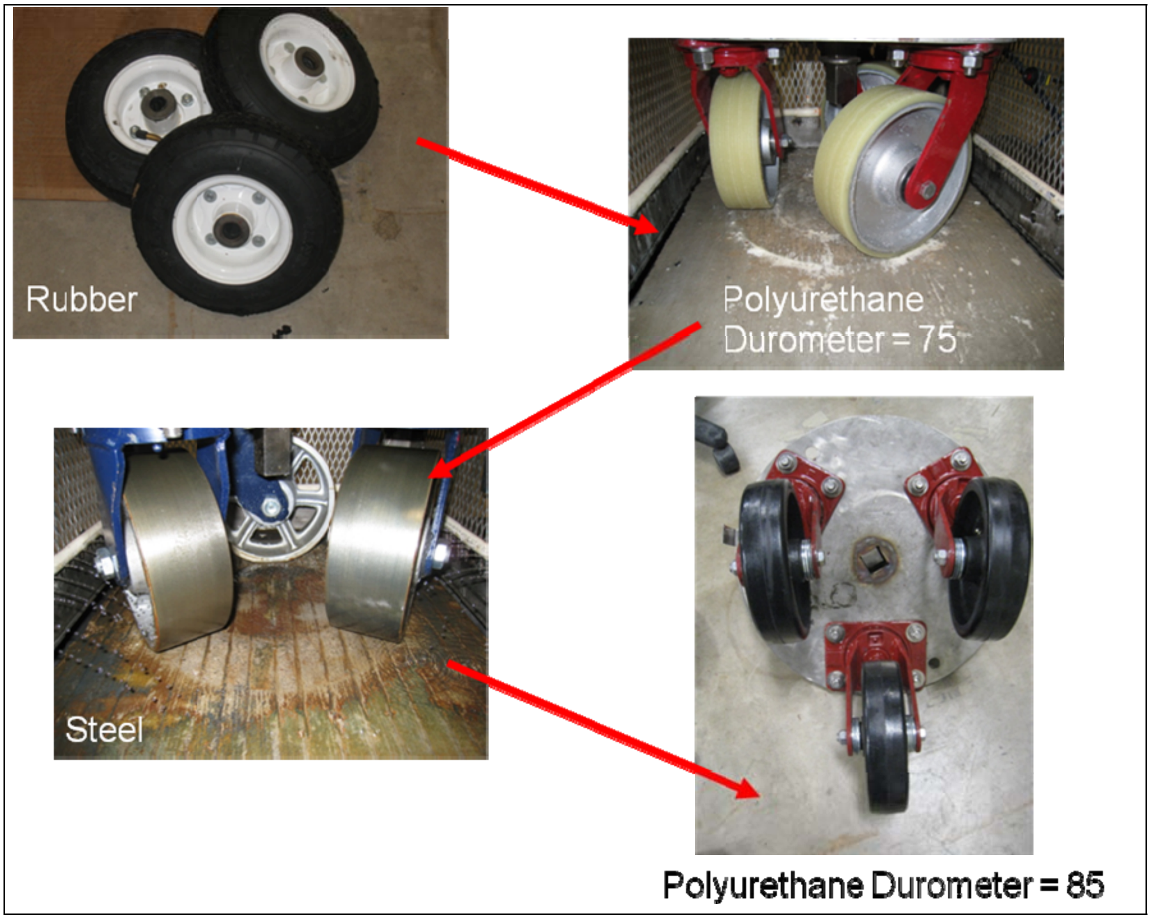

Figure 8.14: Wheels Used on the Polisher

The pneumatic rubber wheels used by NCAT for asphalt did not cause any noticeable wear on the concrete surface after 15,000cycles (Figure 8.15). Using the pneumatic tires on the PCC specimen seemed to have damaged the tires more than it did the concrete. Trying to abrade concrete using pneumatic wheels was judged to be unfeasible because a lot of pneumatic wheels would be needed to wear a single slab; abrading one slab would also would take a long time.

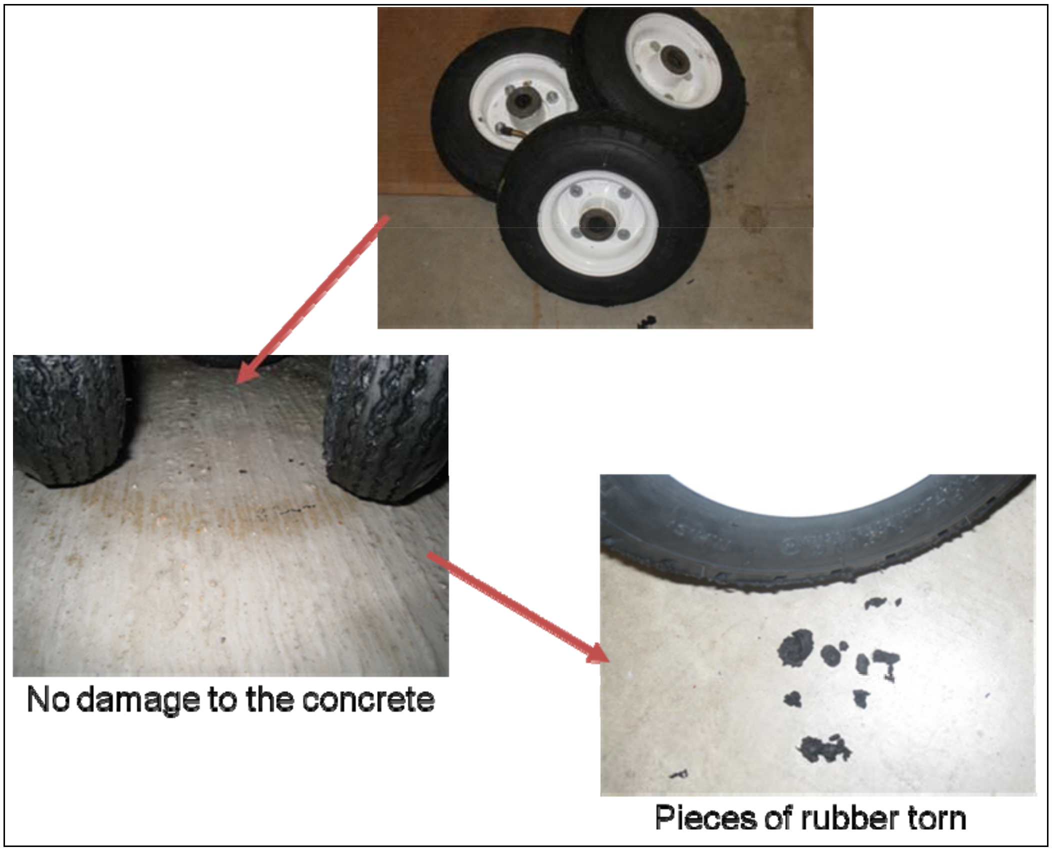

Figure 8.15: Pneumatic Wheels

The second set of wheels used was polyurethane wheels. The polyurethane wheels shown in Figure 8.16 were 2 ½ -in.-wide wheels with a durometer hardness of 75. The polyurethane wheels were able to abrade the surface of the concrete (Figure 8.16); the wear caused by the polyurethane also seemed to resemble wear patterns observed in the field (discussed in Chapter 9). Although the polyurethane wheels were able to wear the concrete surface, the wheels were also severely worn after only 30,000 cycles.

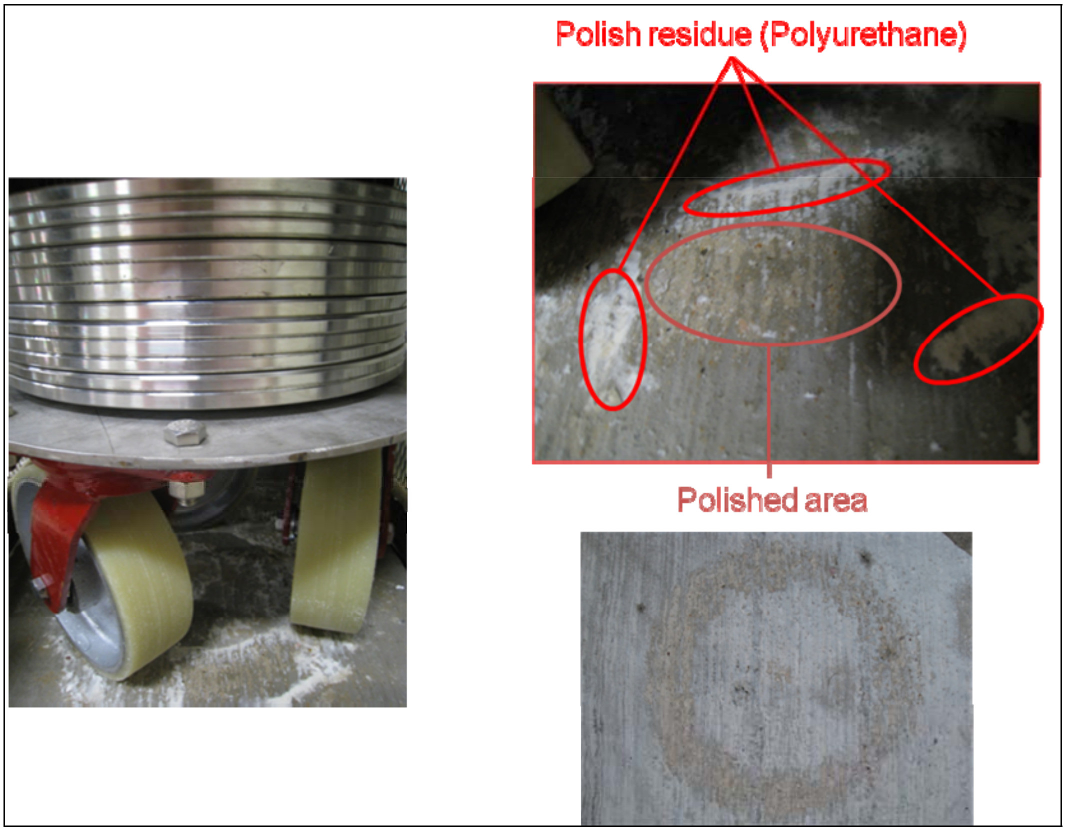

Figure 8.16: Polyurethane Wheels

The last wheel material tested was steel (Figure 8.17). Abrasion using steel wheels was attempted on many different surfaces. The steel wheels caused polishing on all tested surfaces after only a few hundred cycles. The steel wheels polished slabs made with siliceous aggregates as much as they polished surfaces made with limestone aggregates. The wear patterns also did not resemble what had been observed in the field. Polishing with the steel wheels was discontinued because the steel wheels were found to cause excessive wear that did not resemble wear caused by traffic on pavements.

Figure 8.17: Steel Wheels

The best results were obtained using the polyurethane wheels. For this reason a second set of polyurethane wheels was obtained. The black polyurethane wheels (Figure 8.14) were 2-in. wide and had a durometer hardness of 85. Four mortar slabs were abraded using those polyurethane wheels; two were made with siliceous sand (Colorado River Sand), and the other two were made with a limestone MFA (Texas crushed stone). For those slabs friction and texture measurements were taken before the slabs were abraded and after each abrasion cycle using the CTM and DFT. The change in texture measured using the CTM are shown in Figure 8.18. Compared to the slabs made with limestone, the slabs made with siliceous sand had lower MPD values before and after abrading the surface.

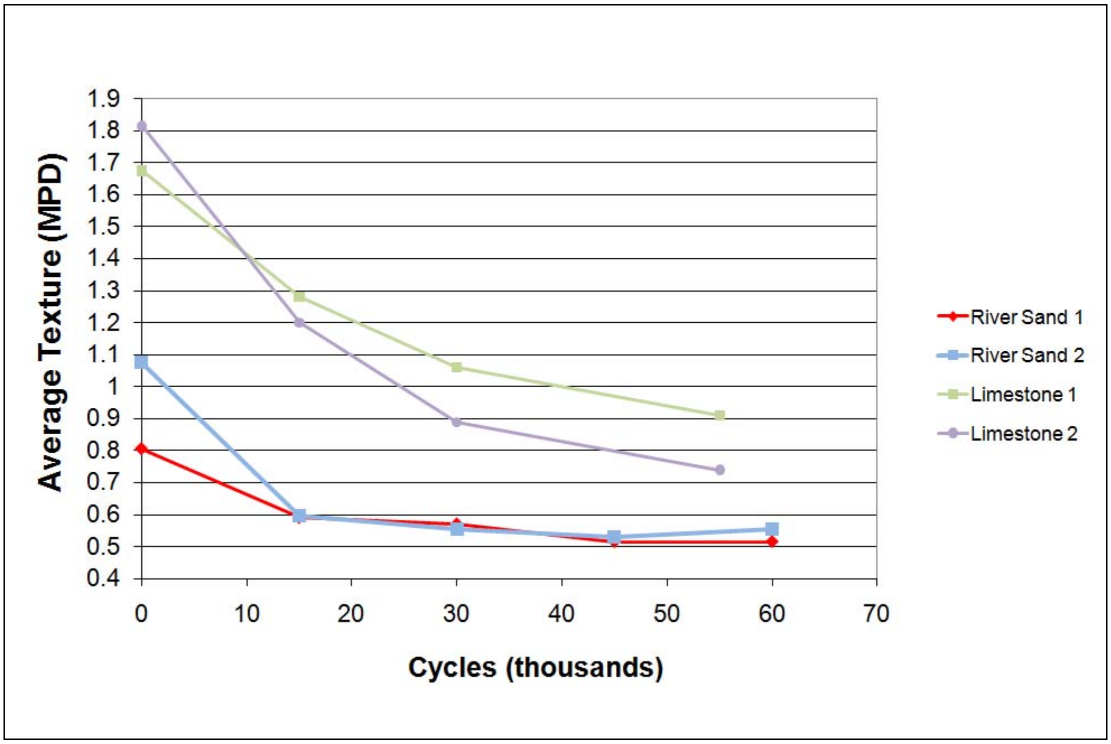

Figure 8.18: Change in Texture Values for the River Sand and limestone MFA

The change in friction values measured by the DFT is shown in Figure 8.19. Although the slabs made with the limestone sand had higher texture (Figure 8.18), the slabs made with siliceous sand had higher friction values. The friction values for the slabs made with siliceous sand remained to be higher than the slabs made with limestone MFA even after 60,000 polishing cycles.

Figures 8.20 shows a picture of a slab made with siliceous sand. The TWPD abraded the wheel path and exposed the siliceous fine aggregates. The exposed fine aggregates were not polished after 60,000 cycles, and the unpolished fine aggregates provided the skid resistance. Figure 8.21 shows a picture of an abraded slab made with limestone MFA. Although the slab seems to still have considerable macro-texture, the top portion of the macro-texture polished; this was what caused the loss in friction.

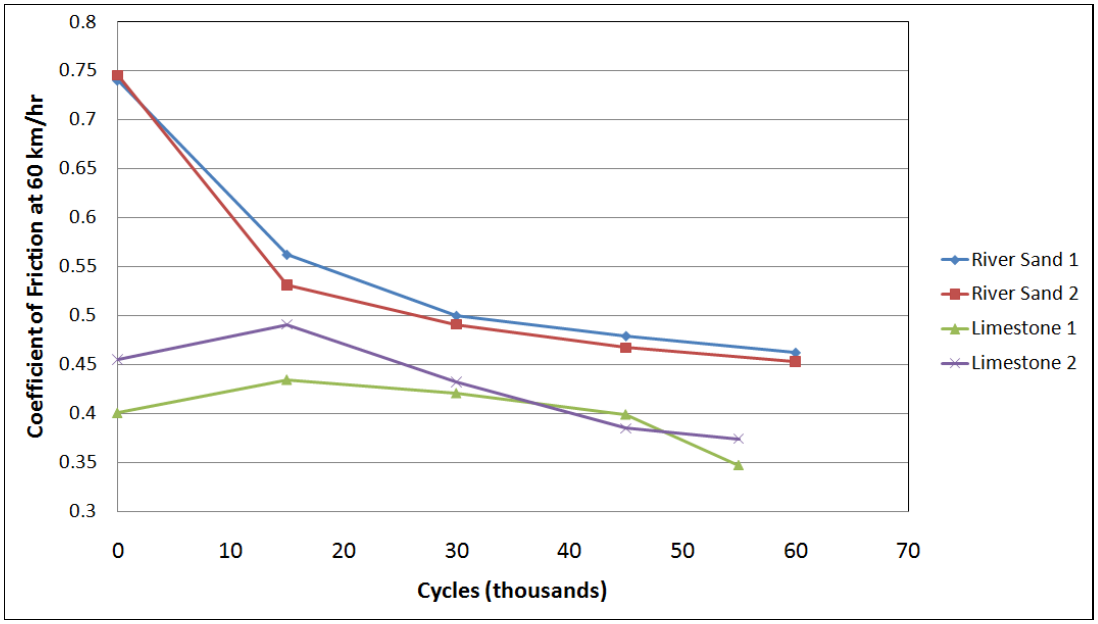

Figure 8.19: Change in Friction Values for the River Sand and limestone MFA

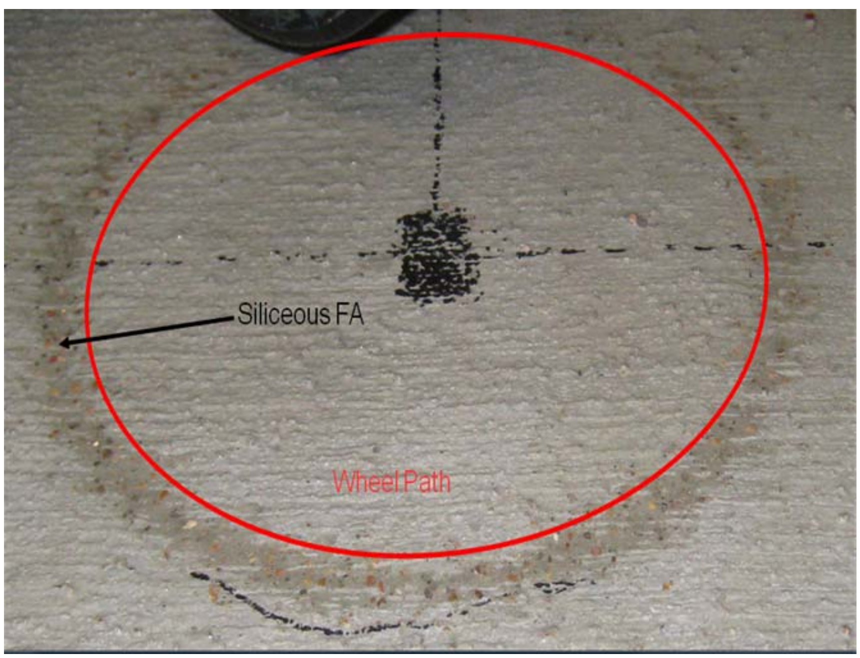

Figure 8.20: Slab made with Siliceous Sand

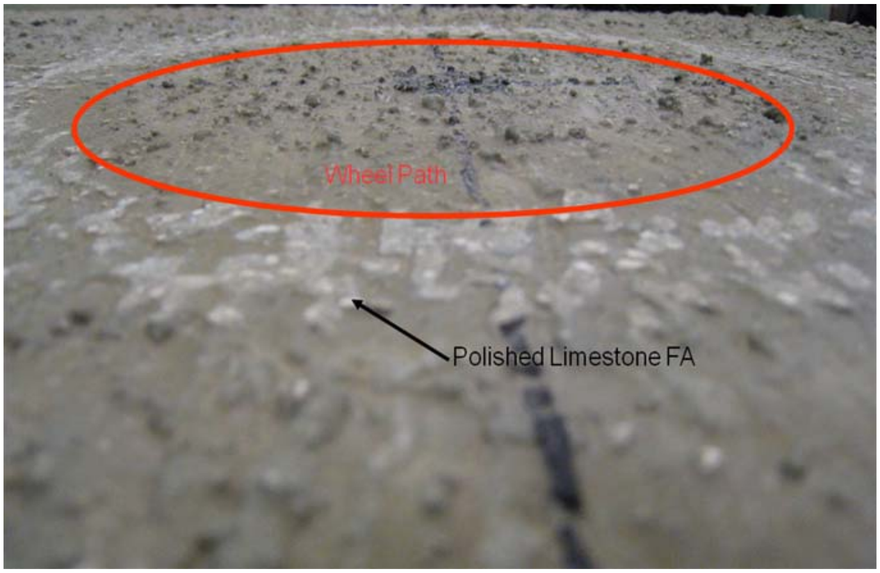

Figure 8.21: Slab made with Limestone MFA

The difference in wear patterns between concrete made with hard and soft fine aggregates is shown in Figure 8.22. Abrasion caused by traffic exposes fine aggregates; harder fine aggregates do not polish and provide frictional resistance, while soft fine aggregates that are exposed polish and cause a drop in frictional resistance.

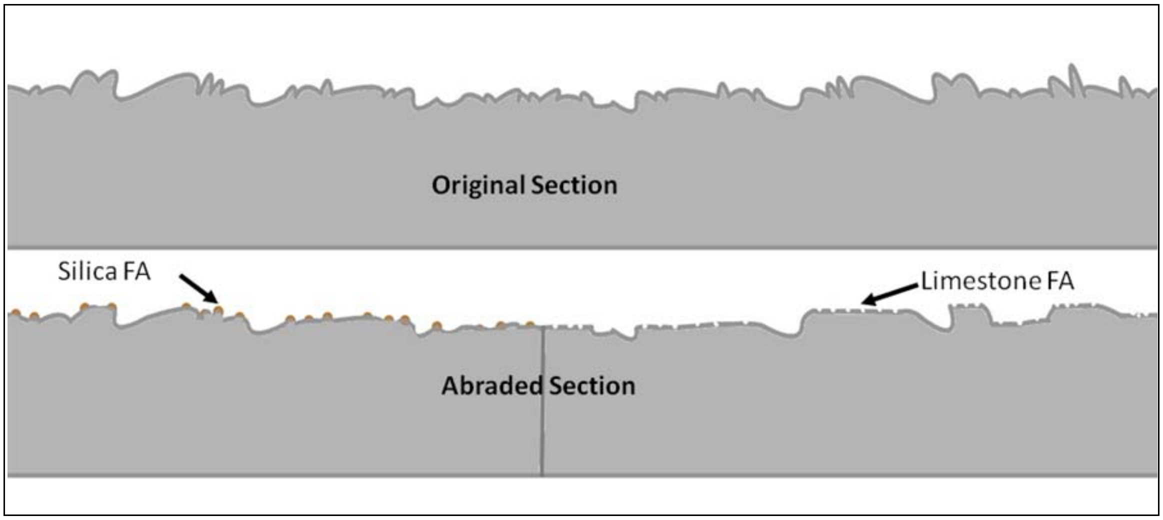

Figure 8.22: Surface made Siliceous Sand vs. Limestone MFA

8.3 CONCLUSIONS

The preliminary testing described in this chapter was necessary to learn how to use the CTM, DFT, and TWPD to evaluate concrete surfaces for skid resistance at the laboratory and in the field. The results obtained from the DFT correlated better with the expected performance of sands. The DFT values did not correlate well with the texture values obtained using the CTM. The macro-texture measured by the CTM is not a good measure of friction; higher MPD values did not always correlate with higher friction values measured by the DFT. The comparison between the trowel-finished surface and the glass surface shows that even concrete surfaces with low macro-texture could still have significant friction. This also showed that the DFT is a better tool for evaluating polished surfaces (the glass surface represents a very polished surface). The materials and setting on the TWPD needed to polish PCC specimen were also investigated. The wheels adopted for the concrete test were polyurethane; the pneumatic wheels used by NCAT for polishing asphalt specimen did not sufficiently abrade the concrete surface like they did on asphalt concrete.

Chapter 9: Field Testing for Skid Resistance

Seven field sections in two different locations were evaluated for skid resistance using a CTM and DFT. Those sections were chosen because they were the only known sections that were made with materials that did not meet the acid insoluble residue (AIR) requirements. The first location had four sections, three of which were constructed with 100% limestone MFA, while the fourth section was a control section made with blended sands. The second location contained three sections made from three different blends of siliceous sand and limestone MFA.

9.1 SECTIONS MADE WITH 100% MFA

9.1.1 Construction of the 100% MFA Sections

The 100% MFA sections were constructed in 2008 as part of a TxDOT implementation project on the usage of MFA containing high microfine content in PCC pavements. The three sections were made with the same source of limestone sand but with varying microfine contents. Information provided in this chapter about the construction of those three test sections was obtained from the report written by McLeroy (2009).

In the manufactured aggregate production process, after rock pieces are conveyed to a crusher, the resulting product is sieved over a No. 4 screen into coarse and fine aggregate. The fine aggregate which is known as the dry screenings is then conveyed to a wet sieving operation. This is done to reduce the fines passing the No. 200 sieve. The wet-sieved product is known as the manufactured sand.

Since the aim of that implementation project was to use sands with varying percentages of microfines, the two types of sands, dry screening and manufactured sand, were combined in different amounts to create fine aggregate blends with 5, 10, and 15% microfine content [McLeroy, 2008]. The gradation of the manufactured sand and the dry screenings are shown in Table 9.1.

	% Retained	
US Sieve	**Manufactured Sand**	**Dry Screenings**
#4	2	2.9
#8	28.1	19.0
#16	29.2	22.4
#30	17.8	16.4
#50	10.5	13.6
#100	5	6.2
#200	1.9	4
Pan	5.5	15.4

Table 9.1: Fine Aggregate Grading [McLeroy, 2008]

The two limestone coarse aggregates used in this implementation project were obtained from the same source as the fine aggregates. The difference between the two aggregates was in their grading; the first was a TxDOT Grade 2 (1 ½-in. maximum size aggregate) and the other aggregate was a TxDOT Grade 4 (1-in. maximum size aggregate). The reason two different coarse aggregates were combined was to obtain better packing of aggregates. Combining a Grade 2 and Grade 4 coarse aggregate is common to the optimized concrete mixtures used in the Fort Worth district. Other materials used on this project included a Type I/II cement (ASTM C 150), a Class C fly ash (ASTM C 618), an air entraining admixture (ASTM C 260), and a water-reducing and retarding admixture (ASTM C 494 Type D) [McLeroy, 2008].

The optimized mixture used for this project is a mixture typically used on TxDOT projects for Class P concrete made with blended sands (Class P is pavement concrete). To create mixtures with microfine contents of 5%, 10%, and 15%, three different blends of sands were used. Table 9.2 shows the batch quantities for each of the three mixtures.

	Concrete Mixtures Proportions *(lb/yd³)*		
	5%	**10%**	**15%**
Cement	362	362	362
Fly Ash	155	155	155
Water	233	233	233
Coarse Aggregate - Grade 2	636	636	636
Coarse Aggregate - Grade 4	1,177	1,177	1,177
Drying Screenings (TXI Bridgeport)	0	684	1,368
Manufactured Sands (TXI Bridgeport)	1,368	684	0

Table 9.2: Concrete Mixture Proportions [McLeroy, 2008]

Laboratory testing for fresh and hardened properties was conducted on each of the three mixtures prior to field implementation [McLeroy, 2008]; these properties included slump (Tex-430-A), unit weight (ASTM C 138), compressive strength (Tex-418-A), modulus of elasticity, abrasion resistance (ASTM C 944), and coefficient of thermal expansion (Tex-428-A). A summary of all the results is shown in Table 9.3. The goal of those tests was to ensure that the performance of the concrete made with limestone MFA can meet TxDOT requirements. The abrasion test was included to show that the addition of microfines does not necessarily reduce abrasion resistance (as implied by ASTM C 33). All properties tested for those three mixtures yielded acceptable results. It should be noted, however, that the concrete was evaluated for abrasion resistance and not for skid resistance. ASTM C 944 is a test that evaluates wear of concrete or mortar by measuring the loss in mass and not the loss in texture or friction. The abrasion/wear test described in

ASTM C 944 better relates to wear caused by the use of studded tires rather than polish caused by traffic.

	Concrete Mixtures		
	5%	**10%**	**15%**
Slump *(in.)*	0.5	2.75	0.75
Unit Weight *(lb/ft^3)*	150	148	151
28-day Compressive Strength *(psi)*	6,370	6,155	6,160
28-day Modulus of Elasticity *(ksi)*	5,320	5,310	5,360
Coefficient of Thermal Expansion *(µstrain/°C)*	5.1	5.1	4.9
Abrasion *(average loss - grams)*	0.9	1.3	1.1

Table 9.3: Laboratory Concrete Tests Results Obtained from McLeroy (2008)

The next part of this project was to implement the mixtures tested at the laboratory and in the field. Paving began on the 5% microfine mixture in July 2008. The first truck delivered to the site had a concrete temperature between 90-95°F (32-35°C) [McLeroy, 2008]. The first truck was rejected, but the high temperatures remained a problem in subsequent trucks [McLeroy, 2008]. The slump measured for the concrete shown in Figure 9.1 was around ¼ -in. which is below the TxDOT requirements of ½ to 2 ½ -in. slump. The paving machine did not have much effect vibrating the low slump concrete that was being delivered. Some of the concrete delivered also had higher workability than what was required (Figure 9.2). This was probably due to the addition of water to the concrete.

Figure 9.1: Low Slump Concrete - 5% Microfine Mixture [McLeroy, 2008]

Figure 9.2: Concrete with a Slump Exceeding the Requirements [McLeroy, 2008]

Placing the concrete was not the only problem encountered during the construction of those sections. The contractor had a very hard time finishing the surface of the concrete because the mixtures were too stiff and lacking mortar on the surface. To resolve this issue, the surface was sprayed with water (Figure 9.3). Enough water was sprayed on the concrete surface to permit it to be finished. Tined and carpet drag finishes were used on the surfaces of all three sections.

Figure 9.3: Finishability Problems Encountered with 100% MFA Sections [McLeroy, 2008]

A fourth section was constructed adjacent to the three sections made with 100% MFA and referred to as the TxDOT optimized mixture. This section was a control section that had 50% siliceous and 50% limestone MFA (Table 9.4). No workability or finishability problems were encountered when this section was cast.

	Concrete Mixtures Proportions *(lb/yd³)*
	TxDOT Optimized
Cement	394
Fly Ash	170
Water	254
Coarse Aggregate - Grade 2	620
Coarse Aggregate - Grade 4	1148
Manufactured Sands (TXI Bridgeport)	667
Siliceous Sand (TXI Paradise)	664

Table 9.4: TxDOT Optimized Mixture Design

Table 9.5 shows the difference between the compressive strength of the concrete made at the laboratory and that of the concrete used for the field sections. The concrete used in the field probably had higher water-to-cement ratio (and that is not even accounting for the surface of the concrete).

	Concrete Mixtures		
	5%	**10%**	**15%**
28-day Compressive Strength - Lab *(psi)*	6,370	6,155	6,160
28-day Compressive Strength - Field *(psi)*	5,480	5,240	4,850
Standard Deviation of Field Compressive Strength *(psi)*	720	230	330

Table 9.5: Lab and Field Compressive Strength

9.1.2 Texture and Friction Evaluation of 100% MFA Sections

Two visits have been made since the sections were constructed in 2008. During those two visits, the texture and friction of the sections were measured using the CTM and DFT; the first visit was in September 2009 and the second was in December 2010. In September 2009 values for sections 1 and 2 were obtained; while in December 2010 all four sections were measured for texture and friction. Figure 9.4 shows that sections 1 and 2 seem to be highly polished on the wheel path. Section 3 and 4 were constructed on the inside lane – thus both those sections were exposed to different traffic. The outside lane (sections 1 and 2) is exposed to more truck traffic, and that is probably why sections 1 and 2 were more polished than section 3.

Figure 9.4: 100% MFA Sections December 2010

Figures 9.5 and 9.6 show the data collected using the CTM and DFT. For each of the four sections, six measurements were taken at three different locations; three measurements on the wheel path, and three between the wheel paths.

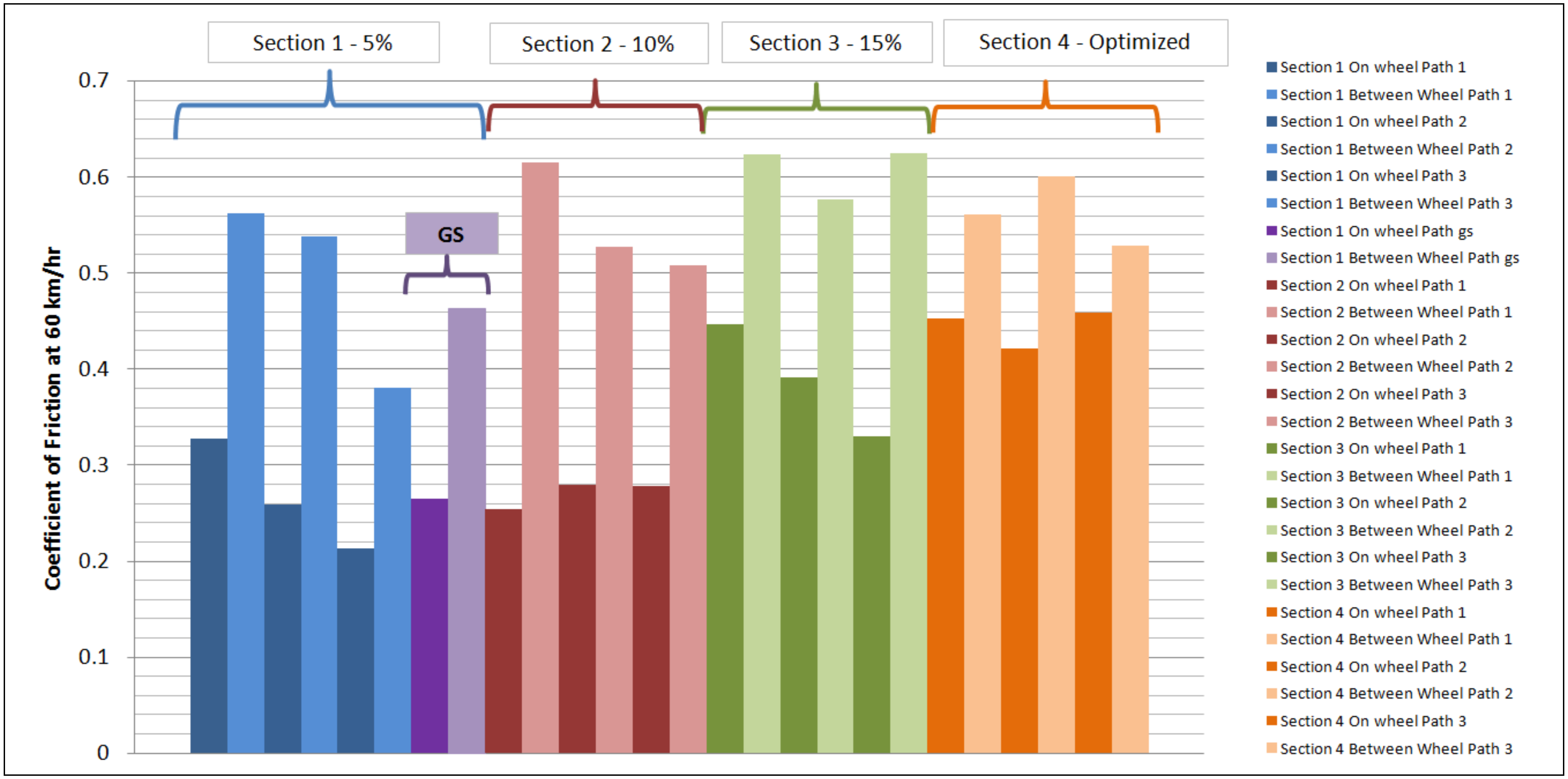

Figure 9.5: Measured DFT60 Values for 100% MFA Sections

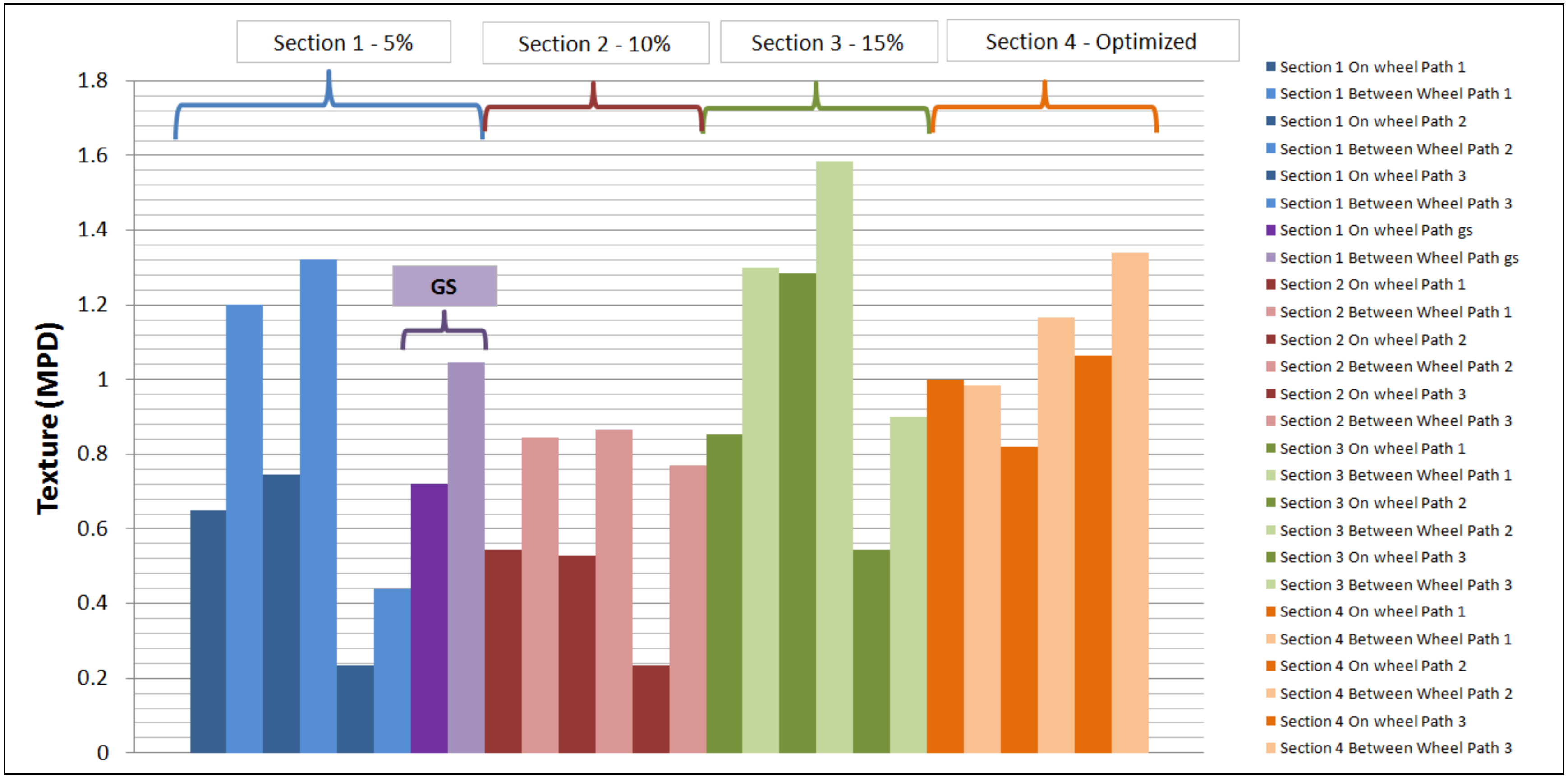

Figure 9.6: Measured CTM Values for 100% MFA Sections

The values obtained between wheel paths are good estimates of the original condition of the pavement before it was subject to traffic. The measurements taken on the wheel path represent the current condition of the pavement (what the vehicles are driving on). Compared to sections 3 and 4, the DFT60 values of sections 1 and 2 on the wheel path were lower. Lower DFT60 values indicate a loss of micro-texture (polishing). The texture values shown in Figure 9.6 show a higher drop in the texture (macro-texture) for sections 1 and 2. The 100% MFA on sections 3 was not abraded as much as sections 1 and 2; this was not due to a difference in materials used but because section 3 was not exposed the same traffic (less truck traffic). Values obtained on section 4 were slightly higher than the values obtained on sections 3.

A fourth measurement was taken on section 1; this was done because a small area in section 1 was diamond ground (Figure 9.7 – GS in Figures 9.5 and 9.6 stands for Ground Surface). Grinding the surface exposed the coarse aggregates; this however did not seem to improve friction or texture values on the wheel path.

Figure 9.7: Ground PCC Pavement (Section 1)

Figures 9.8, 9.9, 9.10, and 9.11 show the difference between the pavement surfaces on the wheel paths and between the wheel paths. Sections 1 and 2 were clearly more abraded and polished than sections 3 and 4 on the wheel path (high loss of macro-texture). The between-the-wheel-path pictures show that all sections had the same finishing.

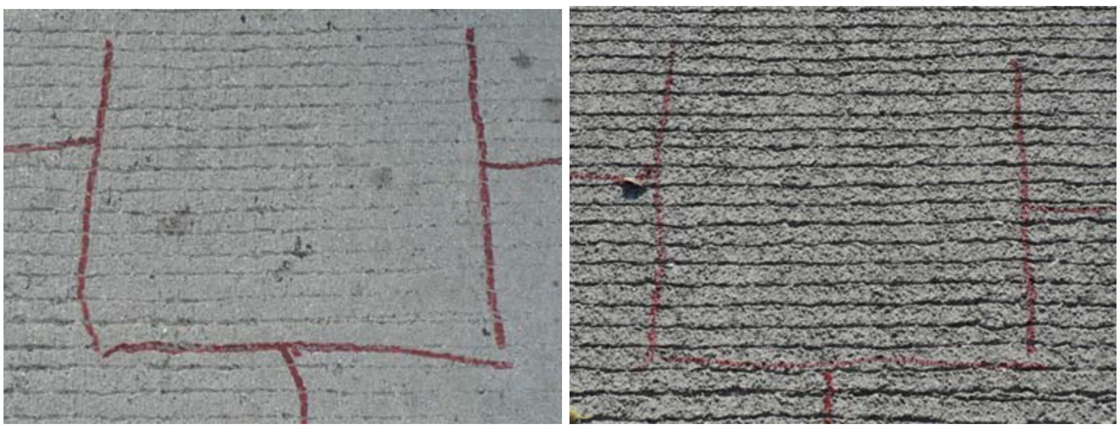

Figure 9.8: Section 1 Wheel path (left) vs. Between Wheel Path (right)

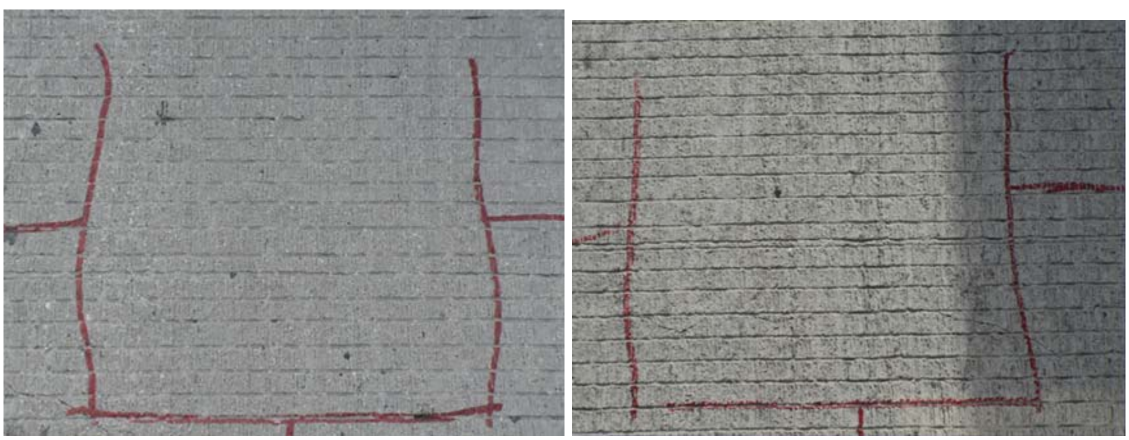

Figure 9.9: Section 2 Wheel path (left) vs. Between Wheel Path (right)

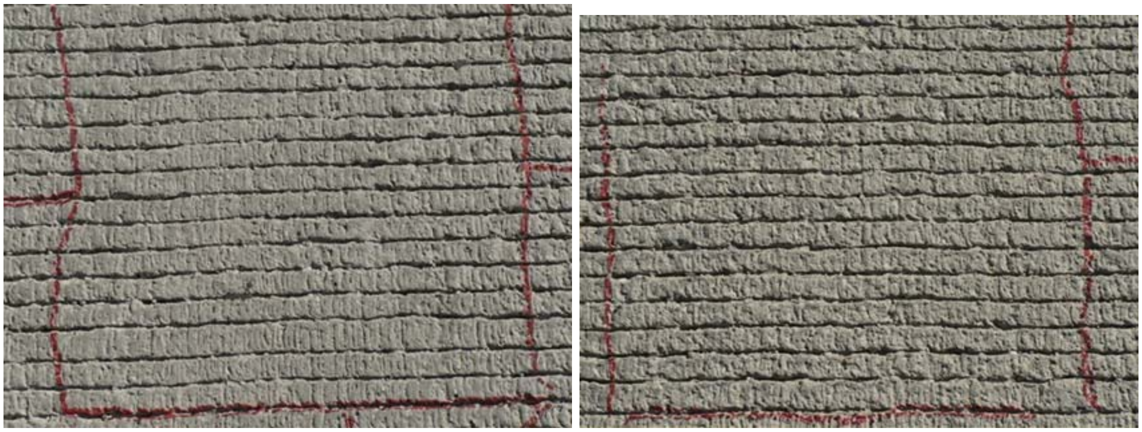

Figure 9.10: Section 3 Wheel path (left) vs. Between Wheel Path (right)

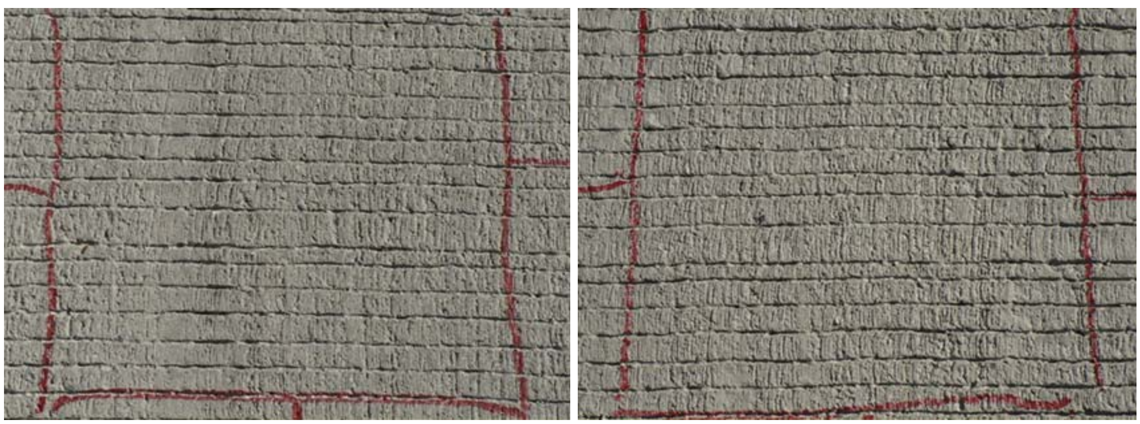

Figure 9.11: Section 4 Wheel path (left) vs. Between Wheel Path (right)

Figure 9.12 shows the average change in DFT60 value since 2008. The values between the wheel path can be assumed to represent near the original condition of the pavement in 2008, while the values obtained on the wheel path in 2009 and 2010 represent the actual condition of pavement when the measurements were taken. Sections 1 and 2 experienced a large drop in DFT60 between 2008 and 2009; the drop in DFT60 between 2009 and 2010 was less significant.

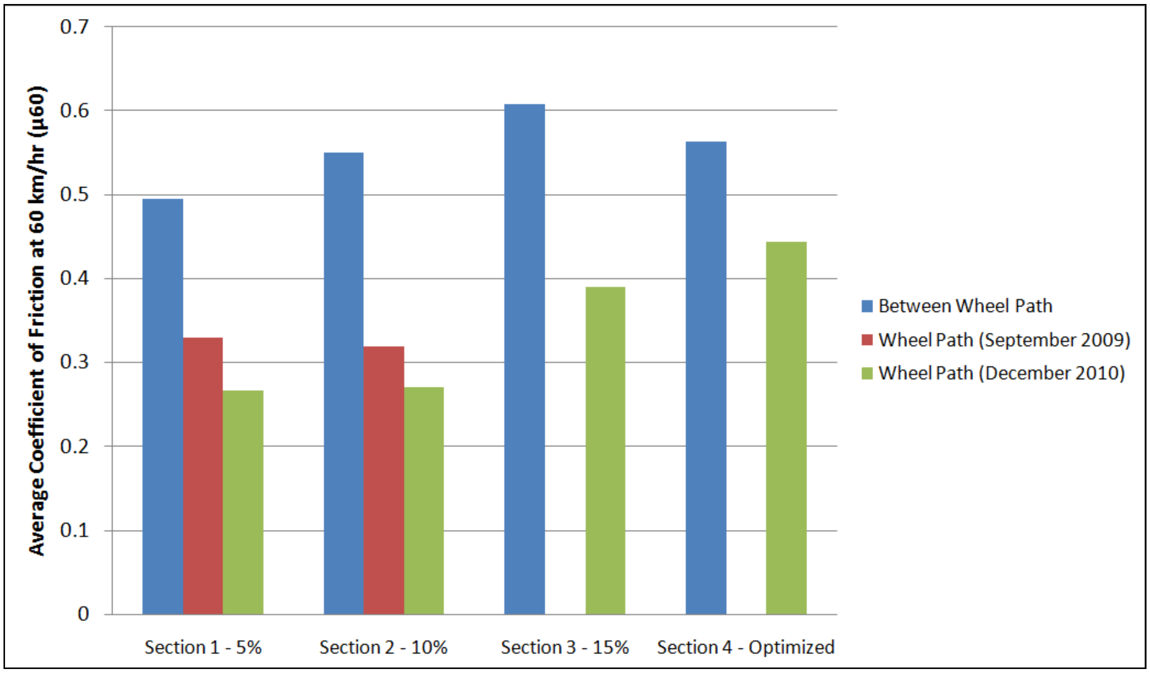

Figure 9.12: Change in DFT60

9.2 Blended Sand Sections

In 1995, three sections were constructed by TxDOT with blends of sands not meeting the TXDOT 60% AIR limit. The three sections were constructed on the inside lane of a highway mainly used by trucks transporting aggregates (the sections are subject to a very high percentage of truck traffic). The following blends of fine aggregates were used for those three sections:

- A 60/40 TXI Paradise (siliceous)/TXI Bridgeport (limestone) blend (AIR = 40%)
- A 50/50 TXI Paradise (siliceous)/TXI Bridgeport (limestone) blend (AIR = 35%)
- A 40/60 TXI Paradise (siliceous)/TXI Bridgeport (limestone) blend (AIR = 29%)

TxDOT regularly evaluates the skid resistance of those blended sand sections. The skid numbers measured by TxDOT in 1997 and 2005 as well as the AIR values are shown in Table 9.6. Note that the measurements were made using a skid trailer with ribbed tires (probably at 40mph).

	Ribbed Tire Average Skid Number (SN)		
	Acid Insoluble Reside (AIR %)	August 1997	February 2005
60/40 Blend	40	43	39
50/50 Blend	35	43	36
40/60 Blend	29	40	35

Table 9.6: Skid Numbers for Blended Sand Sections

The values shown in Table 9.6, show that the 60/40 blended section had the least drop in skid between 1997 and 2005. The sections with 50/50 and 40/60 blends had similar skid values in 2005. Note that all three blended sand sections are exposed to the same traffic.

In December 2010, the site where the blended sand sections are located was visited. Figure 9.13 shows a picture of the section with 60/40 blended sand between the wheel path and on the wheel path. It was hard to visually differentiate between the two surfaces shown in Figure 9.13 (no major loss in macro-texture). Similar observations were made for the 50/50 and 40/60 blended sands sections (The degree of wear and polish cannot be visually distinguished on those blended sections). The CTM and DFT values measured for the blended sand sections are shown in Figures 9.14 and 9.15.

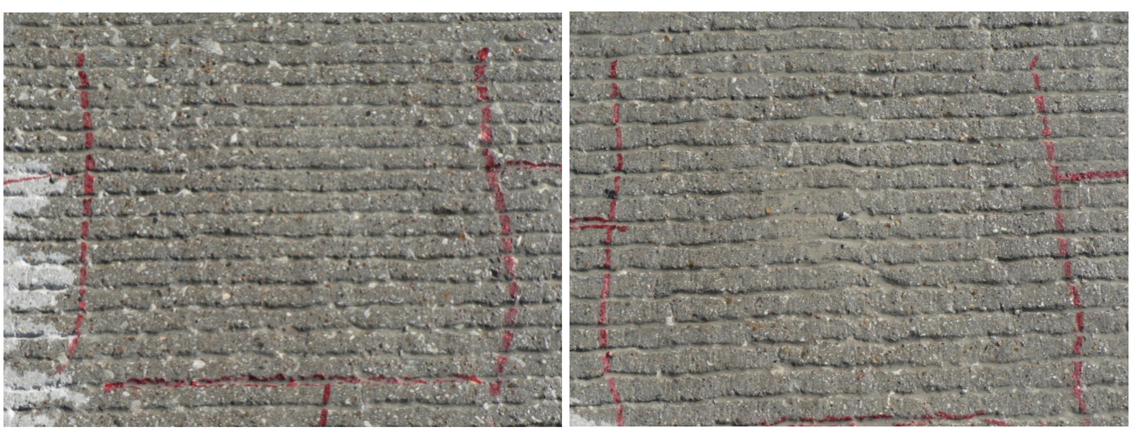

Figure 9.13: 60/40 Blended Section Wheel path (left) vs. Between Wheel Path (right)

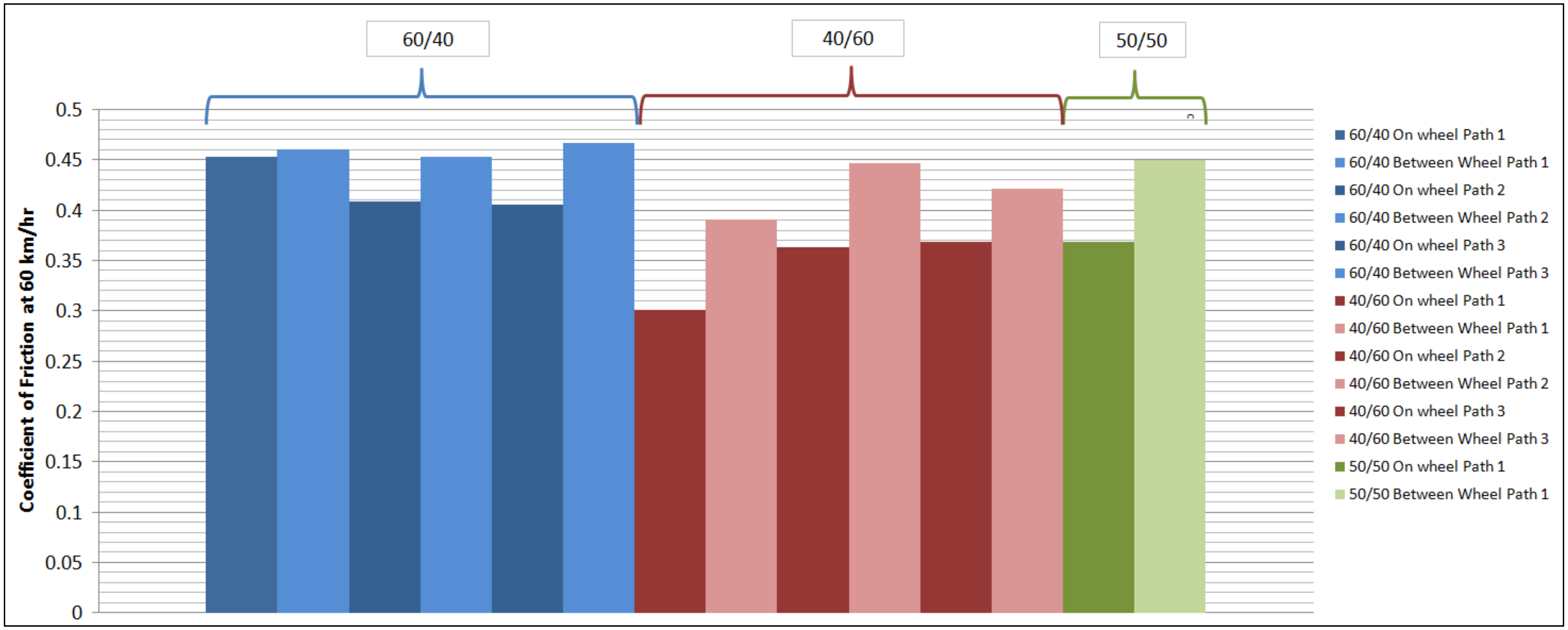

Figure 9.14: Measured DFT60 Values for Blended Sections

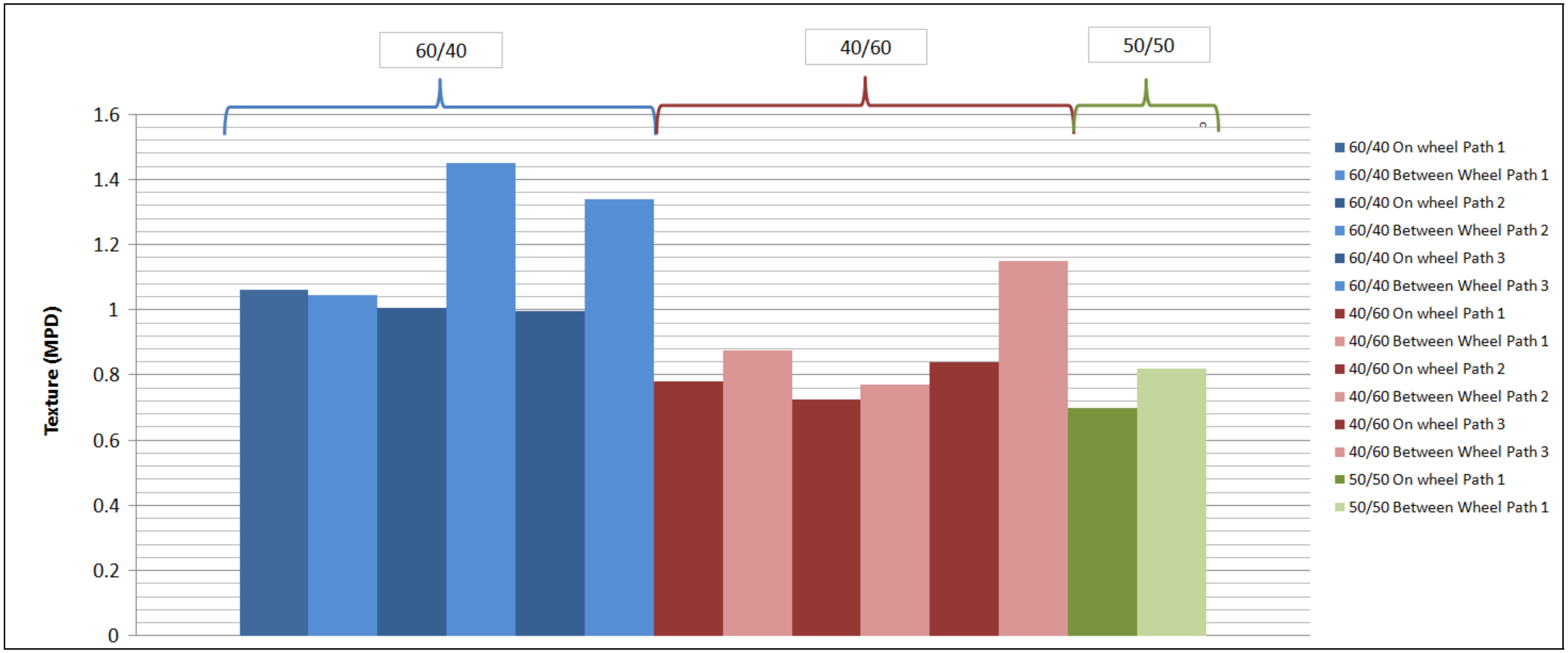

Figure 9.15: Measured CTM Values for Blended Sections

Although only two texture and friction values were measured on the 50/50 blended section, the measured values on that section do not seem to be very different from the values measured on the 40/60 blended sand section. The values measured on the 60/40 section had the highest texture and friction values. Compared to the texture and friction values shown in Figures 9.5 and 9.6 for sections 1 and 2 on the wheel path, the values for the blended sections in (Figures 9.14 and 9.15) were much higher.

9.3 EXCESSIVELY WORN SECTIONS

During the visit to the site where the blended sections were constructed, the TxDOT area engineer, Bill Nelson, identified some highly polished and deteriorated PCC pavements in the area. Photographs of those sections are shown in Figures 9.16, 9.17, and 9.18. The cause of this deterioration could be attributed to the excessive truck traffic in the area where those sections are located. Note that no measurements were taken on those sections because the materials those sections were made up of could not be indentified The pictures shown only serve as an example to show how excessive traffic can cause deterioration of PCC pavements.

Figure 9.16: Excessively Worn Section (1)

Figure 9.17: Excessively Worn Section (2)

Figure 9.18: Excessively Worn Section (3)

9.4 ANALYSIS AND CONCLUSIONS

Using the CTM and DFT values obtained from the two sites visited, the equivalent skid trailer numbers at 40 km/hr were computed. The formulas used to compute the skid numbers were presented in 3.3.4. Figure 9.19 shows the average calculated skid number values (SN) for the 100% MFA (sections 1 and 2) and the three blended sand sections. The SN values shown in Figure 9.19 are the calculated SN values at 40 km/hr using smooth tires. Figure 9.19 shows that even after 15 years of service the blended sand sections have not yet reached the trigger SN value of 20 (defined in 3.3.2). The 100% MFA have already reached an $SN(40)_{smooth}$ value that is lower than 20. The 60/40 blend also seems to have maintained the highest skid number compared to the other two blends that have higher limestone fine aggregate content.

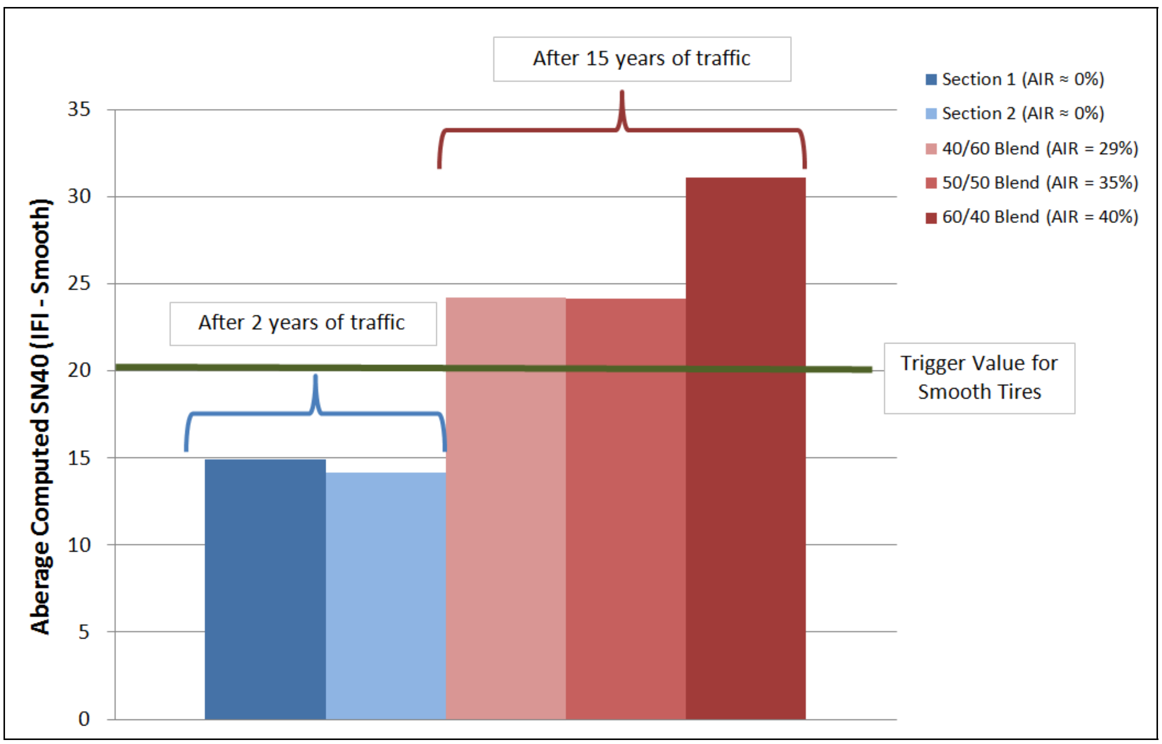

Figure 9.19: 100% MFA vs. Blended Sands (Smooth Tire)

The SN values shown in Figure 9.20 are the calculated SN values at 40 km/hr using ribbed tires. Unlike the smooth tires, ribbed tires are only affected by the micro-texture of the pavement (they are a better way of evaluating the degree of polish of fine aggregates). The blended sand section had values much higher than those of the sections made with 100% MFA. $SN(40)_{ribbed}$ increased as the siliceous content of the pavements increased. The values computed for $SN(40)_{ribbed}$ are very close to the skid values that were measured by TxDOT in 2005 (shown in Table 9.6).

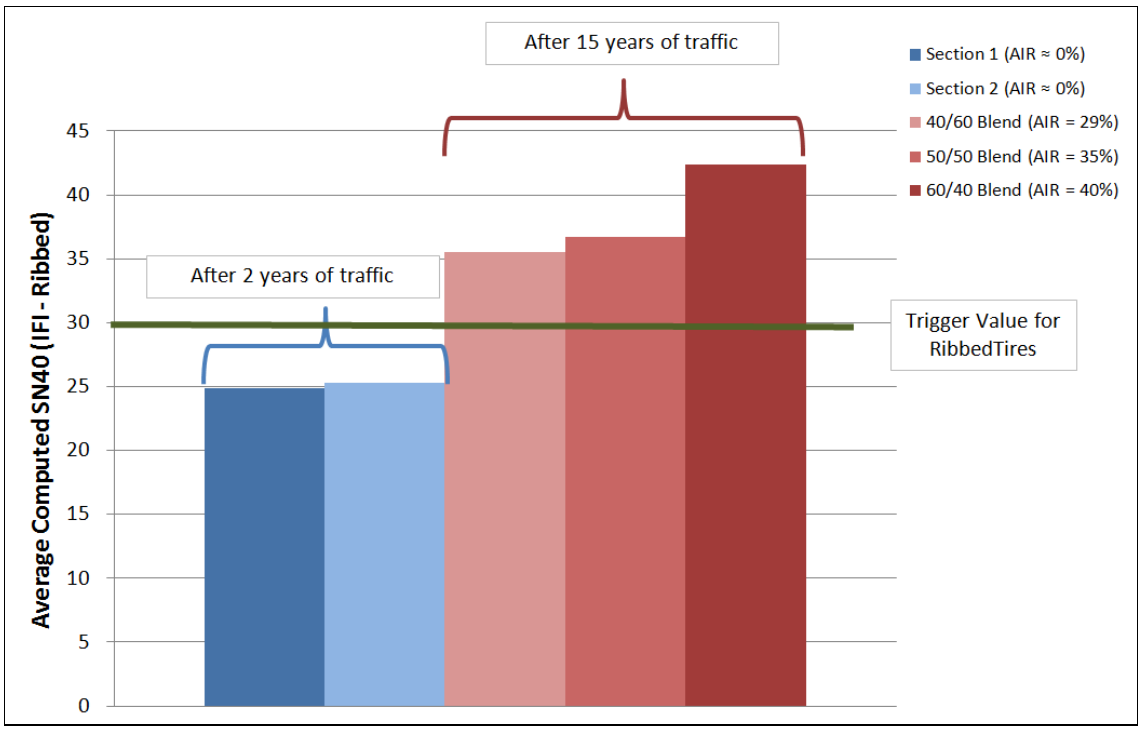

Figure 9.20: 100% MFA vs. Blended Sands (Ribbed Tire)

The values presented in this chapter only compared the PCC pavement sections based on the type of sand that was used, and based on the age of the sections. To be able to do a better comparative analysis of those sections it is important to also compare how much traffic each of those sections is exposed to. Traffic data could not be obtained from TxDOT (was not available), and that is why this type of comparison was not done. Although traffic data were not used for the comparison, the data presented in this chapter are enough to show that there is significant performance difference between PCC pavements made with blended sands and PCC pavements made with 100% manufactured limestone sands. The 100% MFA sections experienced a loss of micro-texture as well as a loss in macro-texture. The loss in macro-texture of sections 1 and 2 could visually be identified without even using a CTM (Figures 9.8 and 9.9). The loss of micro-texture was probably a result of using a limestone sand; the loss of macro-texture could probably be attributed to the presence of a weaker paste at the surface. The water-to-cement ratio of the paste was increased by the excessive addition of water at the surface and that probably resulted in a weaker paste that was more prone to abrasion.

Chapter 10: Evaluation of Hardened Concrete Properties

The effects of aggregates on the hardened properties of concrete were evaluated using standard mixture proportions. In the first section of this chapter the effect of changing fine aggregates on compressive strength, modulus of elasticity, drying shrinkage, and skid resistance were tested for concrete made with different fine aggregates. In the second section of this chapter, the effect of changing concrete mixture proportions for concrete made with MFA was evaluated. The goal of the testing done in the second section of this chapter was to investigate whether or not changing the proportions of a mixture containing carbonate sand influences skid performance.

10.1 MIXING AND TESTING PROCEDURES

The procedures described in ASTM C 192 were used for mixing concrete. However, when MFAs were used, the mixing time had to sometime be increased to insure proper mixing. Also, for the mixtures containing blended sands, the two fine aggregates were added to mixer and mixed prior to the coarse aggregate. This was done to insure that the fine aggregates were well blended (each of the fine aggregates was batched separately). As discussed in chapter 4, a mid-range water-reducing admixture was used to facilitate casting the specimen. The admixture content was varied depending on the fine aggregate and proportions used. The admixture dosage was not recorded since this type of admixture is not usually used in paving concrete; data obtained on admixture content or workability was considered to not be useful.

The compressive strength of concrete was tested at 7 and 28 days following ASTM C 39 procedures using 4-in. x 8-in. cylinders. The modulus of elasticity at 28 days was measured using the procedures described in ASTM C 469 (two cylinders were tested). The method described in ASTM C 157 was used to measure drying shrinkage for

112 days after curing. The concrete tested for shrinkage was not cured for 28 days as specified by ASTM C 157; it was only cured for 7 days. This was done because 7 days of curing better represents curing done in the field (three samples were tested).

Skid resistance was evaluated using the CTM, DFT, and TWPD. Two slabs measuring 20 in. wide and 3 ½ in. deep were tested for each mixture. The change in texture and friction was monitored over 160,000 polishing cycles (3 days of testing per slab). Measurements were taken initially and after 5,000, 40,000, 100,000, and 160,000 polishing cycles. To evaluate the same polished area, each slab was marked so that readings could be taken at the same location (Figure 10.1). All slabs were finished using a broom finish and the surface was cured for at least 28 days before the slabs were tested. Two texture readings were measured using the CTM for each slab at each polishing interval. When measuring friction using the DFT, ASTM E 1911 reports that standard deviation on the same test surface for DFT60 is 0.038, for this reason friction measurements using the DFT at 40,000, 100,000, and 160,000 cycles, were repeated several times on the same slab at the same location until the difference between the last two readings was less or equal than 0.01. The last measurement obtained (usually the lowest) was reported.

Figure 10.1: Typical Markings on a Slab

The black polyurethane wheels described in Chapter 8 were used to polish all slabs tested. Each set of polyurethane wheels lasted about 500,000 cycles (1 wheel per slab). Because those wheels were used, another modification to the TWPD was needed. A vibration dampener was added to TWPD after the TWPD failed several times (Figure 10.2). The failure happened because the wheels used were much stiffer than the original pneumatic wheels that the TWPD was designed for. The stress caused by the wheels on the concrete surface was estimated to be around 50psi (based on the total load and the contact area).

Figure 10.2: Modified Three-Wheel Polishing Device

10.2 Evaluating the Effect of Fine Aggregates on Hardened Concrete Properties

10.2.1 Mixture Proportions

One standard concrete mixture was used to evaluate all fine aggregates tested (Table 10.1). The reason this was done was because the effect of changing mixture proportions on skid resistance was not well understood at that time. The mixture was a 6-

sack mixture with a water-to-cementitious ratio of 0.42 and a sand-to-aggregate ratio of 0.37.

Materials (Volume %)			
Cementitious	Water	Fine Aggregate	Coarse Aggregate
10.73	14.20	27.06	46.01

Table 10.1: Mixture Proportions used for evaluating Fine Aggregates

Table 10.2 shows the combinations of fine and coarse aggregates used for this testing. The choice of coarse aggregates was not expected to influence the skid resistance of the slabs being tested since the surface of the concrete was composed of mortar (cement paste and fine aggregate). Moreover, both coarse aggregates were limestone aggregates obtained from the Bridgeport area.

Fine Aggregate	Coarse Aggregate
Capital Marble Falls	Hanson Perch Hill
Hanson Servtex	Hanson Perch Hill
Texas Crushed Stone	Hanson Perch Hill
Hanson Perch Hill	Hanson Perch Hill
TXI Bridgeport	Hanson Perch Hill
Lattimore Stringtown	Hanson Perch Hill
Colorado River Sand	TXI Bridgeport
Eagle's Nest	TXI Bridgeport
TXI Paradise	TXI Bridgeport
TXI Beckett	TXI Bridgeport
Granbury	TXI Bridgeport
Ingram Rainbow	TXI Bridgeport
Lattimore Cleburne	TXI Bridgeport
Lattimore Rosser	TXI Bridgeport
Trinity Kopperl	Hanson Perch Hill
Trinity Kopperl/Perch Hill Blends	Hanson Perch Hill
TXI Paradise/TXI Bridgeport Blends	TXI Bridgeport

Table 10.2: Combinations of Fine and Coarse Aggregate Used

10.2.2 Siliceous Sands vs. Manufactured Sands

The hardened concrete properties of sands obtained from different sources are compared in this section. The results for the compressive strength at 7 and 28 days are shown in Figures 10.3 and 10.4 The average standard deviation between three compressive strength tests performed was 110 psi. All mixtures reached a compressive strength higher than 6000psi after 7 days of curing. Except for the mixture made with Texas Crushed Stone MFA, the compressive strength of concrete made with the different sands was more or less equal at 7 days. The mixture containing Texas Crushed Stone MFA reached a compressive strength of about 8000psi in 7 days. This might have occurred because Texas Crushed Stone has high microfine content. Research done by Fowler et al, (2008) has shown that higher microfine content could lead to higher compressive strengths. Also, compressive strength is mainly controlled by water-to-cement ratio, and although all mixtures were designed to have the same water-to-cement ratio, the moisture corrections done were influenced by the values of absorption obtained. Rogers and Dziedziejko (2007) found that when using ASTM C 128 for measuring absorption, the presence of microfines results in greater multi-laboratory variation than obtained with the same group of laboratories when the fines are removed. The absorption value obtained for Texas Crushed Stone might not be representative of the real absorption capacity of that aggregate. After 28 days of curing, all concrete mixtures reached a compressive strength higher than 7500psi. The mixture made with Texas Crushed Stone reached a compressive strength of around 9000psi.

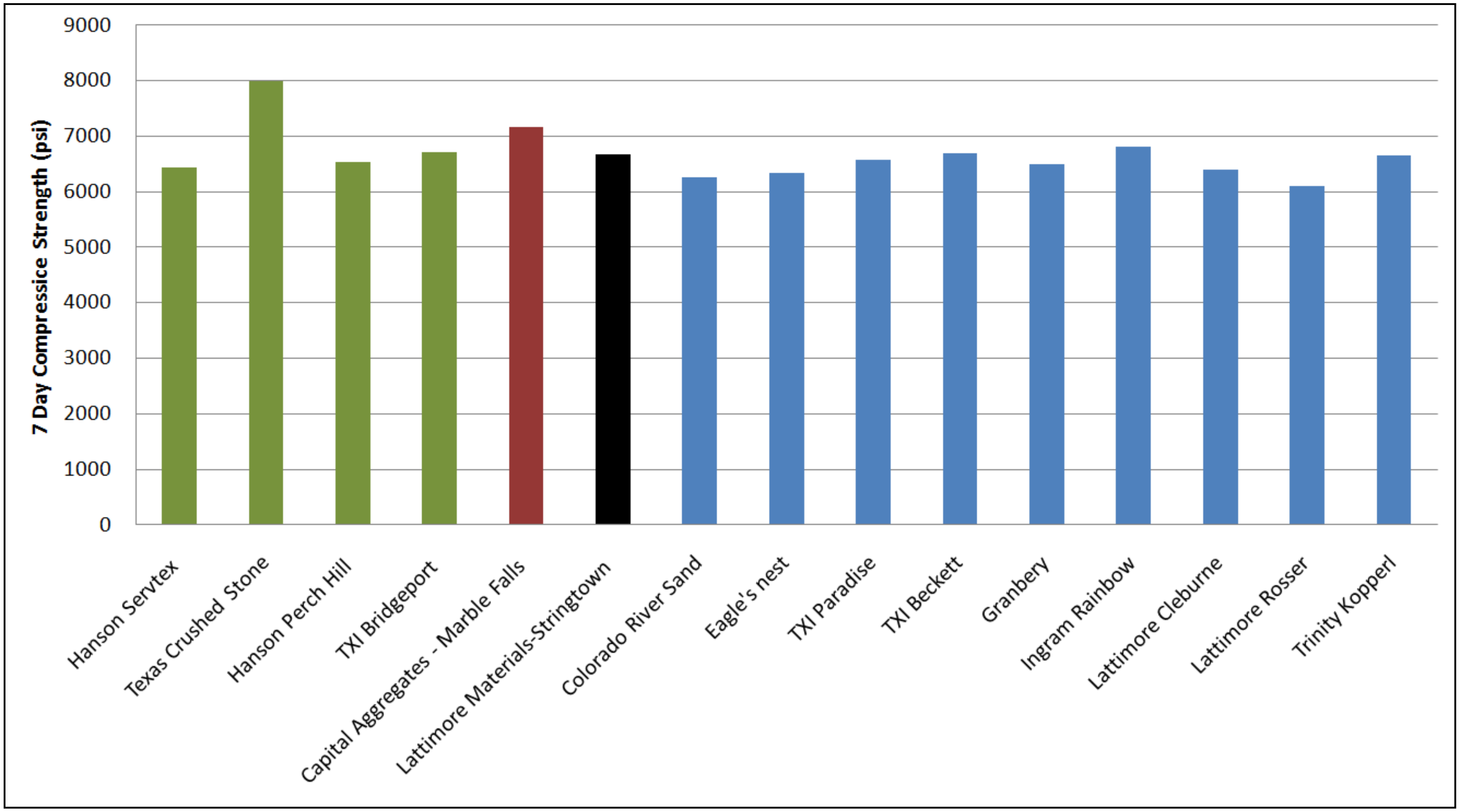

Figure 10.3: Compressive Strength of Concrete made with Different Sands after 7 days of Curing

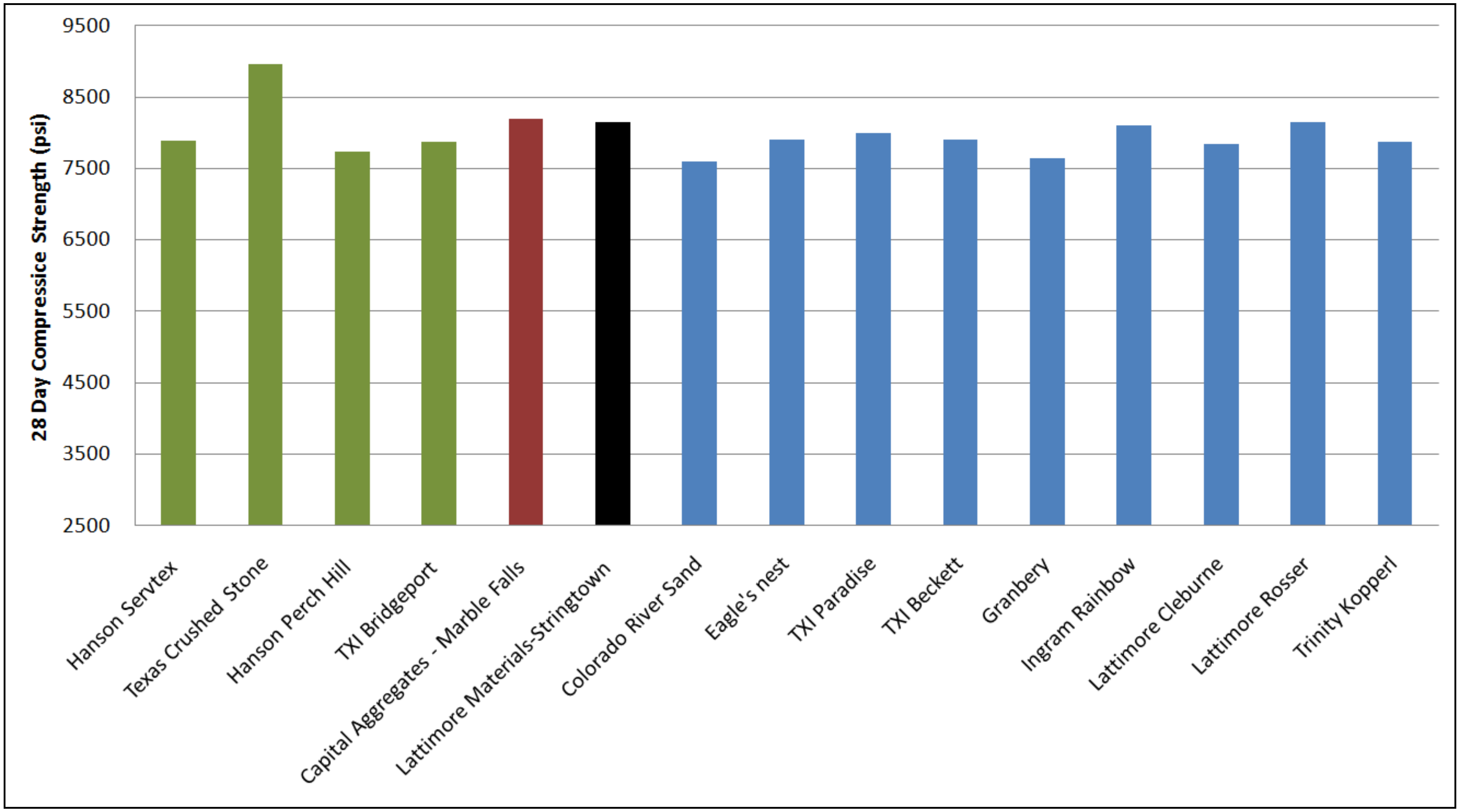

Figure 10.4: Compressive Strength of Concrete made with Different Sands after 28 days of Curing

Results for the modulus of elasticity tested at 28 days are shown in Figure 10.5. The two aggregates that resulted in a significantly higher modulus of elasticity were Capital Marble Falls and Trinity Kopperl. The modulus of elasticity of concrete made with MFA did not otherwise differ from concrete made with siliceous sand.

Drying shrinkage was monitored for all specimens for 112 days. The 112-day shrinkage results are shown in Figure 10.6. Using Lattimore Stringtown in concrete resulted in the highest shrinkage. Lattimore Stringtown had a mineralogy and shape that differed from all other aggregates tested, and that might explain why the shrinkage values obtained using Lattimore Stringtown were different. All other aggregates resulted in shrinkage values that ranged between 300 to 460 μstrain. The use of manufactured carbonate fine aggregates did not have a negative effect on the shrinkage of concrete.

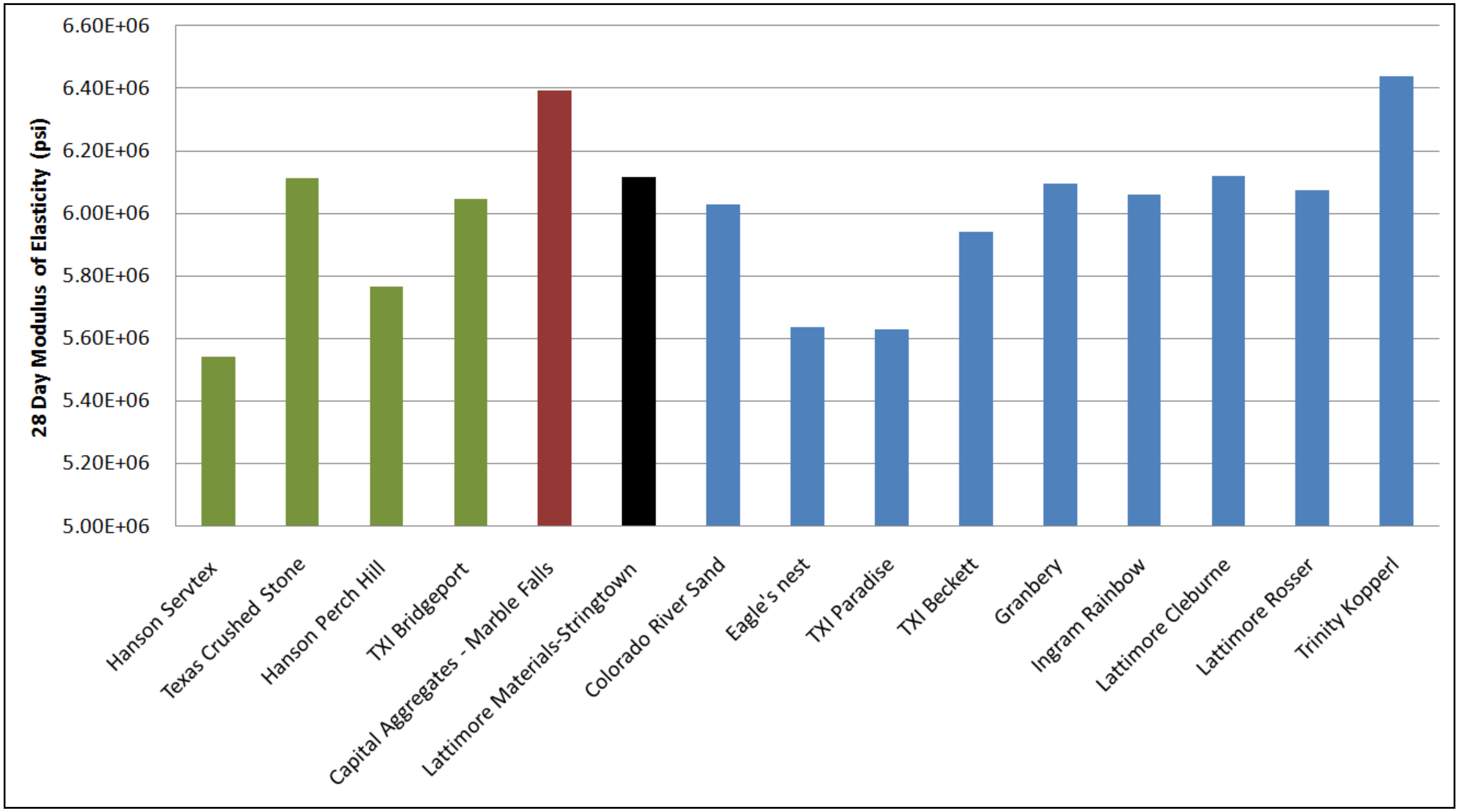

Figure 10.5: Modulus of Elasticity of Concrete made with Different Sands after 28 days of Curing

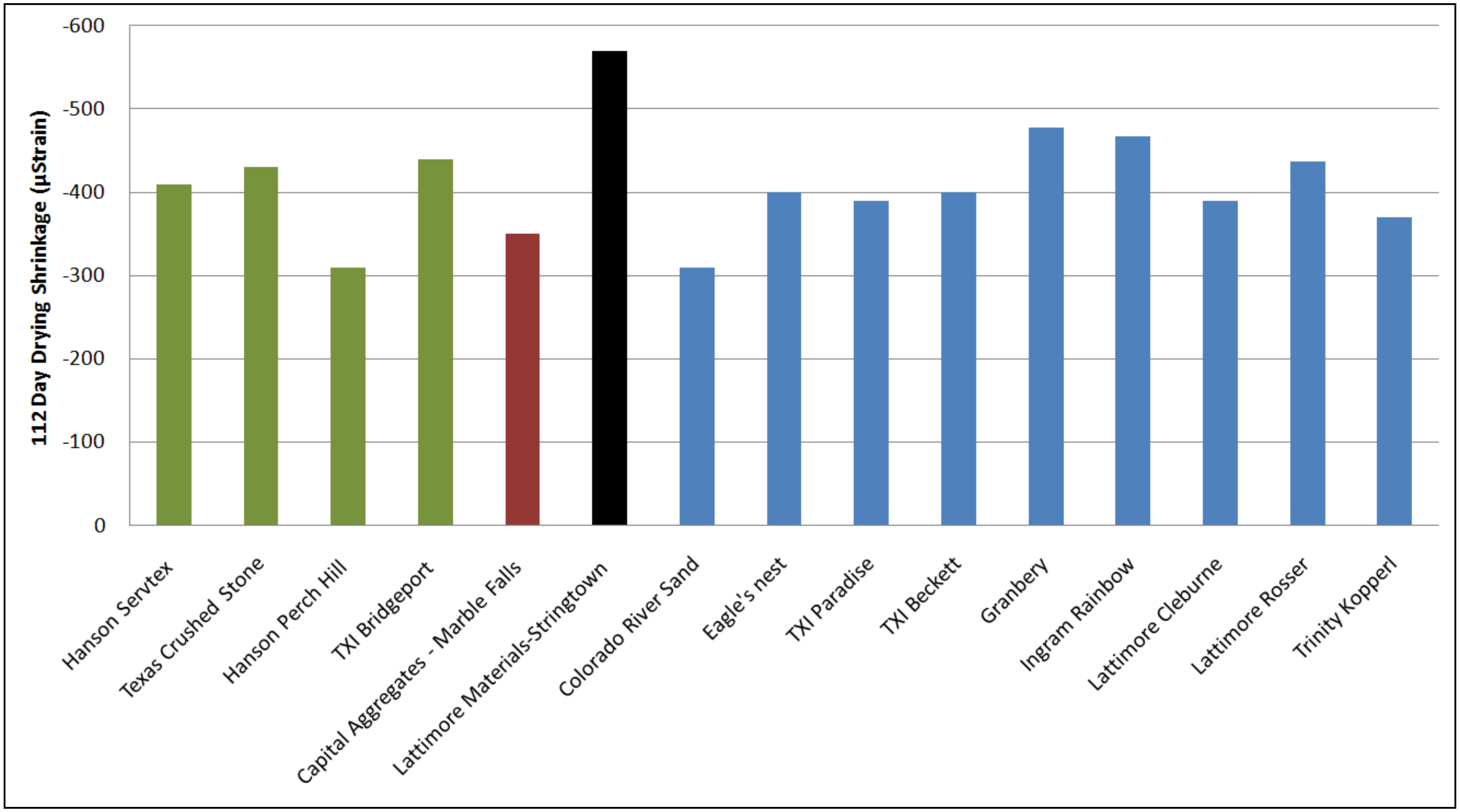

Figure 10.6: Drying Shrinkage of Concrete made with Different Sands

Texture and friction was measured on concrete slabs made with different fine aggregates using the procedures described in 10.1. The average results for the coefficient of friction at 60 km/hr (DFT60) are shown in Figure 10.7 while the average results for texture are shown in Figure 10.8. The average difference in MPD between two slabs made from the same material was 0.155 while the difference in DFT60 values was 0.0212. Except for the slabs made with Lattimore Cleburne, the initial coefficient of friction value at 60 km/hr obtained on each of the surfaces made with siliceous sands ranged from around 0.72 to 0.82. The DFT60 value after 160,000 polishing cycles for all the siliceous sands was higher than a coefficient of friction μ of 0.45. After 160,000 cycles, all siliceous sands had DFT60 values that ranged from 0.47 to 0.52.

The MPD values obtained from finishing the surfaces (initial MPD) ranged from 1.3 to 2.05 for all finished surfaces. After only 5,000 polishing cycles, the MPD was reduced to a range of about 0.7 to 1.2. The only slabs that maintained significantly higher MPD values were the slabs made with the Colorado River Sand. Those slabs had higher MPD values because their initial texture was higher; this texture however did not seem to contribute to an increase in friction after 160,000 polishing cycles. The reduction in texture between 40,000 cycles and 160,000 cycles was not significant compared to the reduction in texture that occurred after the initial 5,000 cycles.

There were no trends between texture and friction results; while the friction values at 160,000 cycles for all siliceous sands converged to a range of 0.47 to 0.52, the range of texture values was wider (0.55 to 1). Also, between 40,000 and 160,000 cycles, the drop in friction was more significant than the drop in texture.

Note that the reason many siliceous sands were tested was to test how sands having different acid insoluble residue (AIR) values above 60% differed. The values obtained in this chapter will be compared to AIR in Chapter 11.

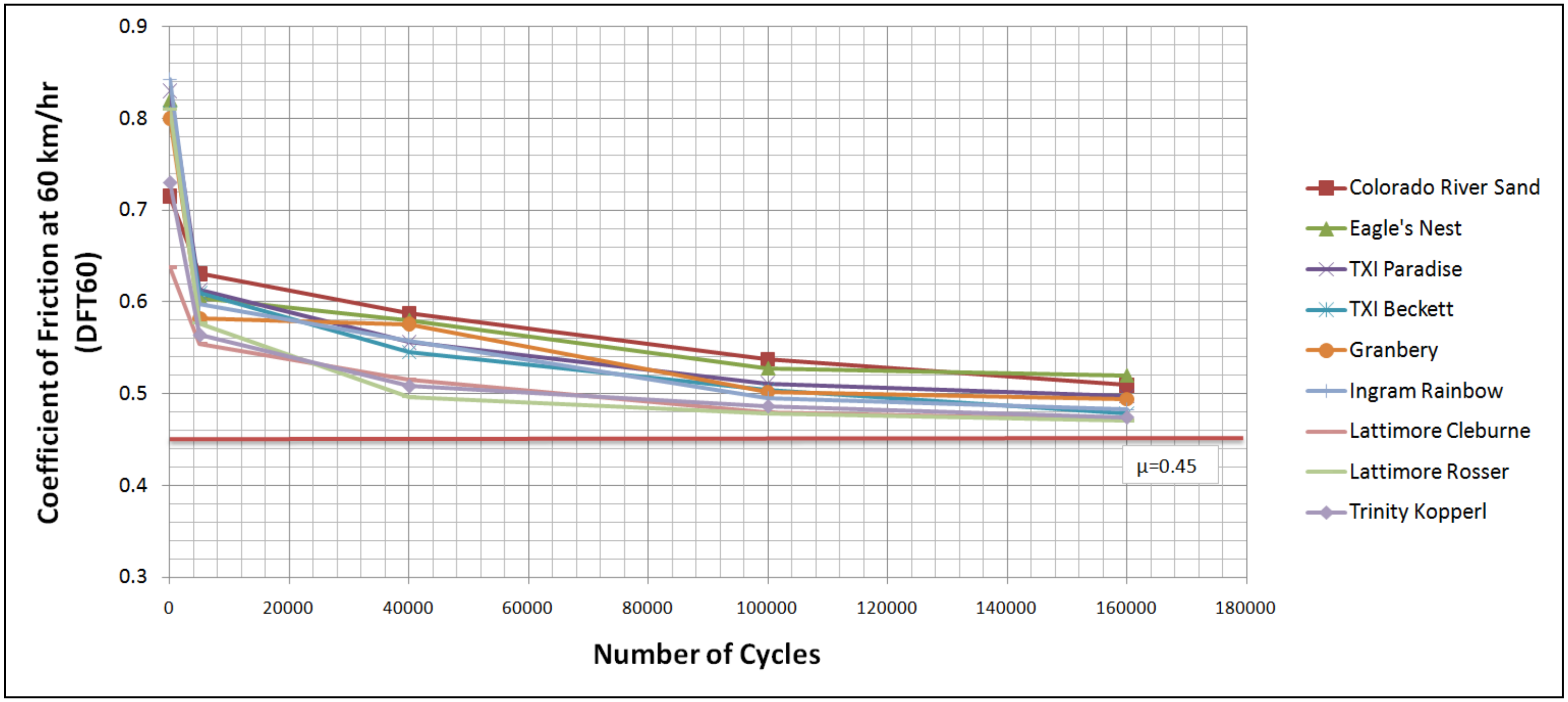

Figure 10.7: DFT60 Results for Siliceous Sands

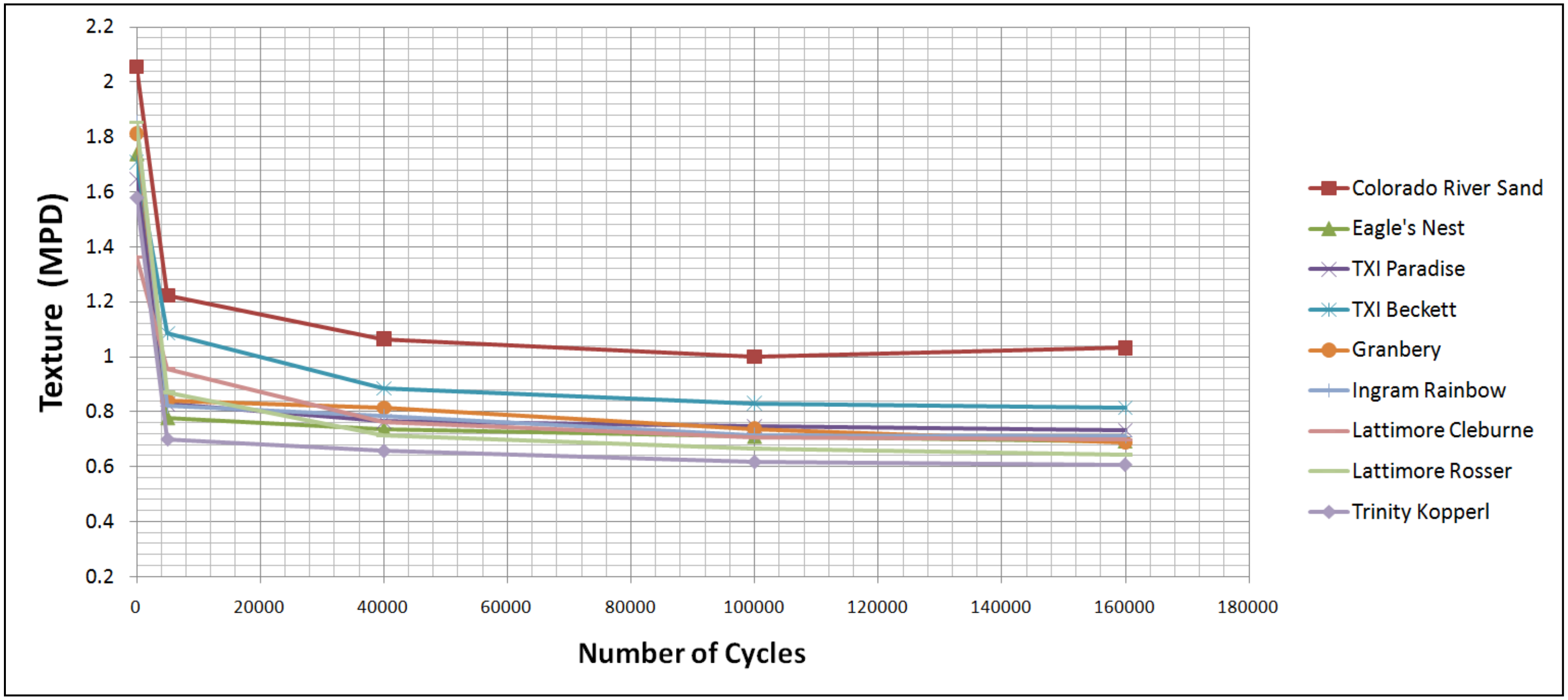

Figure 10.8: Texture Results for Siliceous Sands

The results of the friction measurements for the manufactured sands are shown in Figure 10.9. The average difference in DFT60 values between two slabs made from the same material was 0.0262. The initial finished slabs had DFT60 values ranging from 0.52 to 0.85. Compared to the siliceous sands, the values at 160,000 cycles for the manufactured sands were significantly different; they ranged from 0.37 to 0.5 (the siliceous sands ranged from 0.47 to 0.52). Moreover, the slabs that had the highest initial friction did not necessarily maintain it after 160,000 cycles. Lattimore Stringtown started with a DFT60 value of 0.52 and reached a value of 0.48 after 160,000 cycles. Texas Crushed Stone started with an average DFT60 value of around 0.74; this value dropped to 0.37 after 160,000 cycles. The starting friction value for the manufactured sands did seem to affect the final value at 160,000 cycles. The only three manufactured fine aggregates that had a DFT60 value higher than 0.45 were Lattimore Stringtown, Capital Marble Falls, and Hanson Servtex. The values obtained for Hanson Servtex were not expected; Hanson Servtex is a limestone that has shown poor performance in asphalt concrete and was therefore expected to perform as the other three limestone fine aggregates did.

After the preliminary testing for skid was performed (discussed in Chapter 8), testing the slabs for 160,000 cycles was considered to be adequate to differentiate between different fine aggregates. After the results for the slabs made with Servtex were obtained, it was decided to test three slabs made with carbonate manufactured sands for an additional 340,000 cycles (a total of 500,000 cycles). The DFT60 results are shown in Figure 10.10. The DFT60 values for the slab made with Capital Marble Falls (dolomite) did not change between 160,000 cycles and 500,000 cycles, while the DFT60 values for the slabs made with the Hanson Servtex (limestone) and Texas Crushed Stone (limestone) dropped.

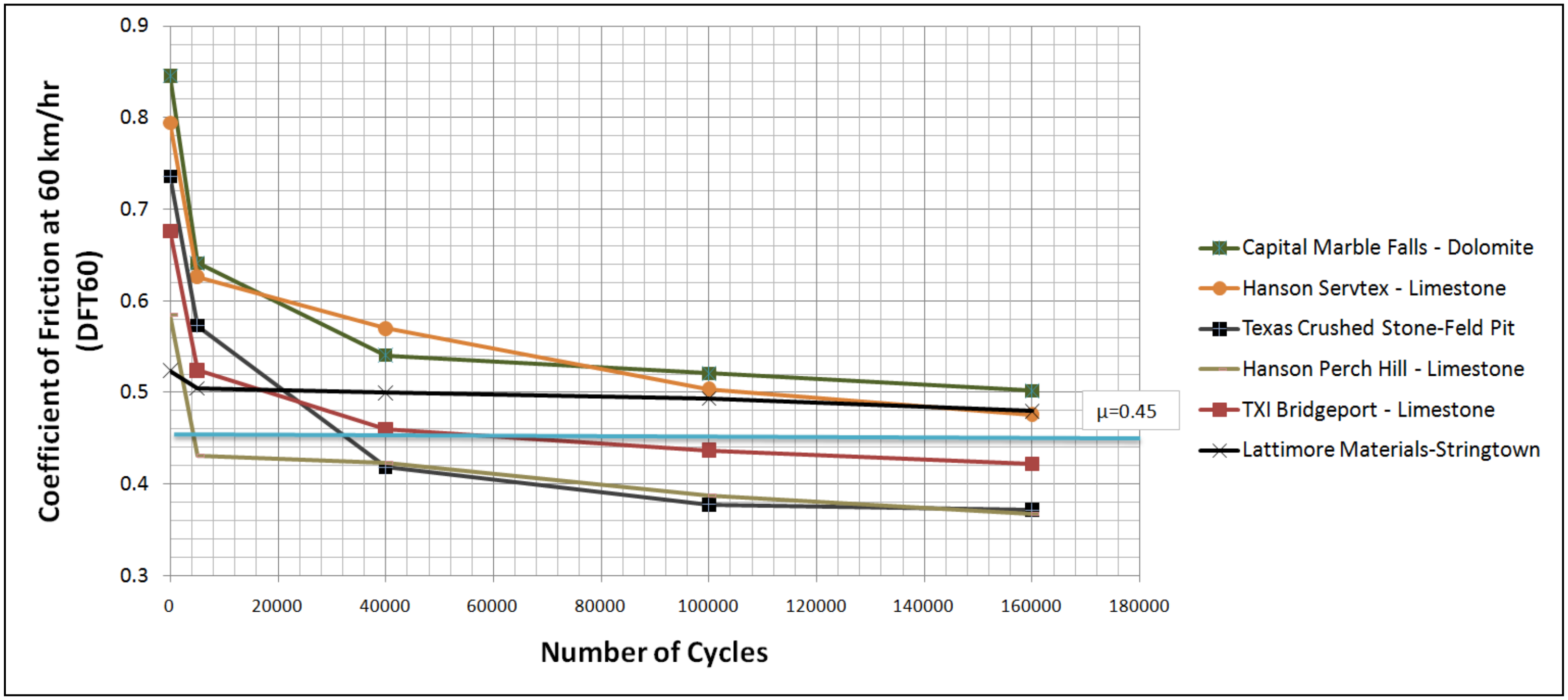

Figure 10.9: DFT60 Results for Manufactured Sands

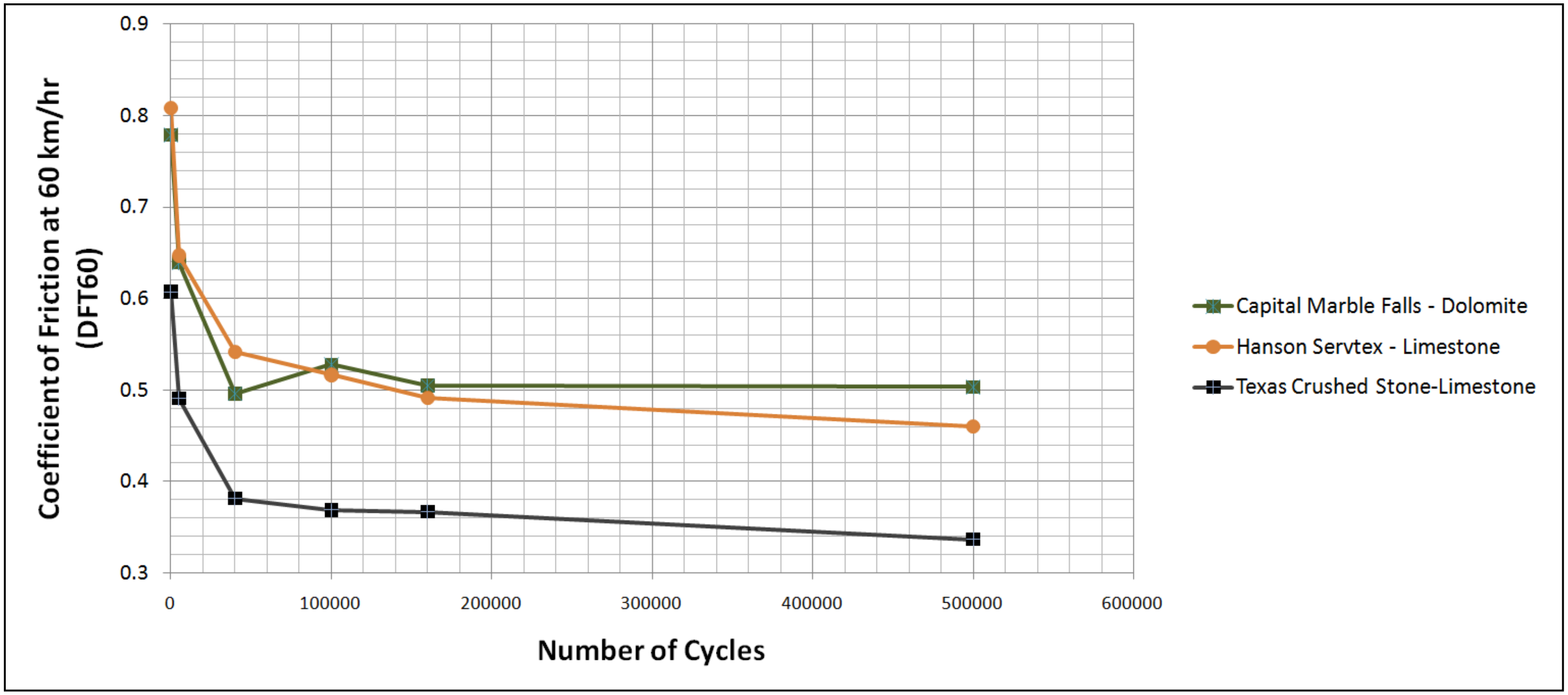

Figure 10.10: DFT60 Results for Manufactured Sands Tested for 500,000 Cycles

The texture results for the concrete slabs made with manufactured sands are presented in Figure 10.11. All manufactured sands had an initial texture ranging from an MPD of 1.7 to 1.9. The average difference in MPD between two slabs made from the same material was 0.148. In general, the initial MPD values obtained with the siliceous sands were lower than the initial MPD values obtained using manufactured sands. This might have occurred because using MFA resulted in having more poorly shaped aggregates at the surface and harsher mixtures. The highest MPD value obtained after 160,000 cycles was for the slabs made with TXI Bridgeport. The lowest MPD value obtained was for Hanson Perch Hill.

The texture values after 500,000 cycles for the three slabs made with carbonate aggregates were also evaluated. The results are shown in Figure 10.12. Although Capital Marble Falls maintained the same DFT60 value at 160,000 cycles, the MPD value at 160,000 cycles was not maintained after 340,000 cycles. Both Texas Crushed Stone and Hanson Servtex also did not maintain their texture values after the additional 340,000 cycles.

Note that only three slabs for each mixture were tested for 500,000 cycles. The reason that more slabs were not tested for 500,000 cycles was because 160,000 cycles seemed to be sufficient to differentiate between slabs made with different sands. Moreover, testing for 500,000 cycles takes 7 days for each slab tested, so testing more slabs for that many cycles is unfeasible if such testing is not necessary.

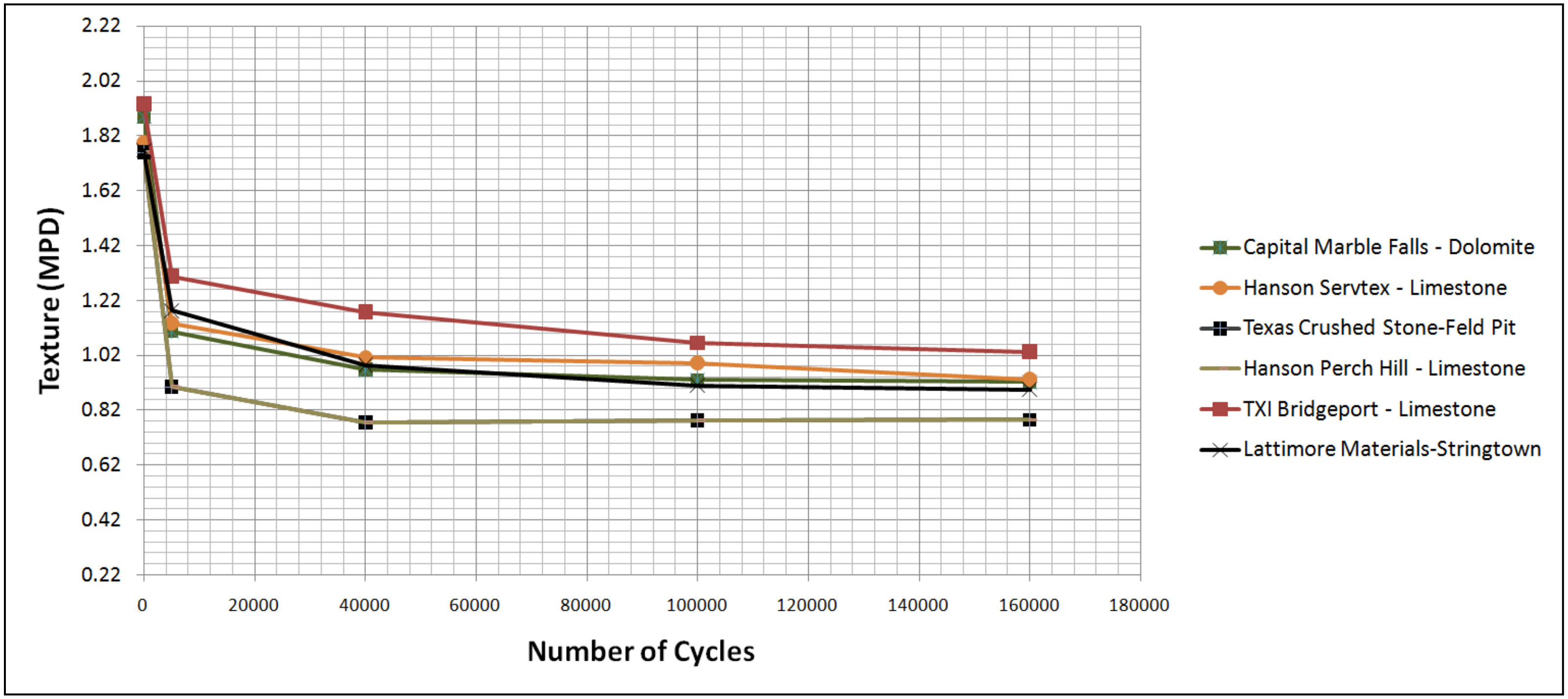

Figure 10.11: Texture Results for Manufactured Sands

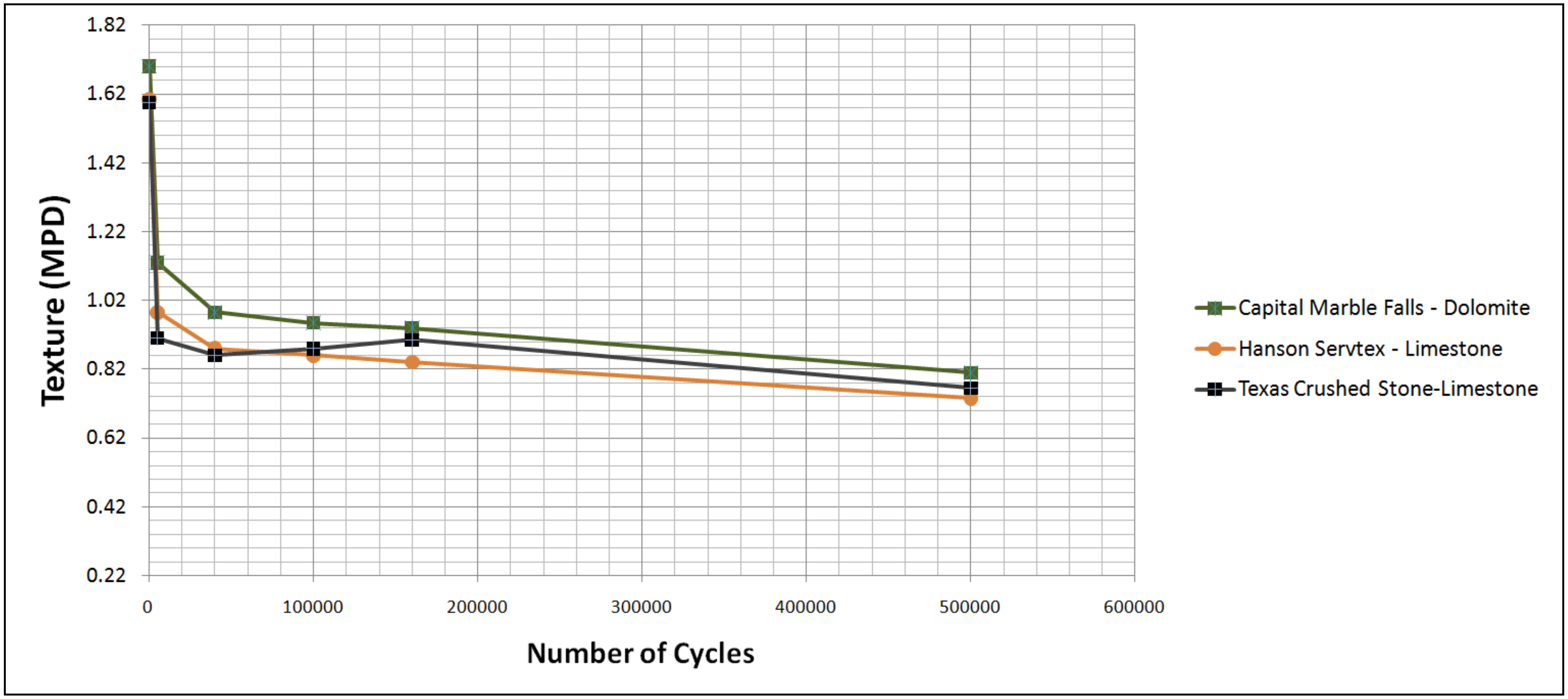

Figure 10.12: Texture Results for Manufactured Sands Tested for 500,000 Cycles

The DFT60 and the MPD results at 160,000 cycles for all the tested sands are presented in Figures 10.13 and 10.14. The average difference in DFT60 values between two slabs made from the same material at 160,000 cycles was 0.006. Except for Hanson Servtex, all limestone sands (green bars) had DFT60 values lower than any of the siliceous sands (blue bars). Because the DFT evaluates micro-texture, those values were expected (field performance test have shown that siliceous sands have better skid performance compared to limestone sands). Capital Marble Falls (dolomite) had a DFT60 value at 160,000 cycles that was comparable to the values obtained with siliceous sands. Unlike Hanson Servtex, Capital Marble Falls was expected to perform better than the other carbonate aggregates. Laboratory results obtained by Balmer and Colley (1966) also showed that dolomitic sands had higher wear indices compared to limestone sands (section 3.2.3.2). However, it is not clear whether or not such performance could be obtained in the field. TxDOT could not identify any field sections made with 100% dolomite sand because dolomite sands do not meet the acid insoluble residue (AIR) limit of 60% and for this reason dolomite sands have not been used at 100% replacement for PCC pavements in Texas.

Lattimore Stringtown was the only other MFA that had good laboratory performance; Lattimore Stringtown meets current AIR requirements but is not used for pavement concrete in the Fort Worth District because it does not meet the organic impurities limit. The Lattimore Stringtown sample used for this project was tested for organic impurities (using ASTM C 40) and the aggregate passed the test. Moreover, the compressive strength obtained using this sand was not lower than the compressive strength obtained using other sands (organic impurities are believed to cause a reduction in strength). The other reason Lattimore Stringtown is not used in PCC pavement is related to its shape. As discussed in Chapter 6, Lattimore Stringtown has a very poor shape which results in poorly workable and finishable concrete.

The expected performance of fine aggregates did not relate well with the texture results obtained in Figure 10.14. The average difference in MPD between two slabs made from the same material at 160,000 cycles was 0.111. The MPD values for all siliceous sands except for the Colorado River Sand at 160,000 cycles were equal or lower than the MPD values obtained for the concrete slabs made with manufactured sands. Poor correlation between macro-texture values obtained using the CTM and micro-texture values obtained using the DFT was expected after the preliminary work on CTM and DFT was done (discussed in Chapter 8).

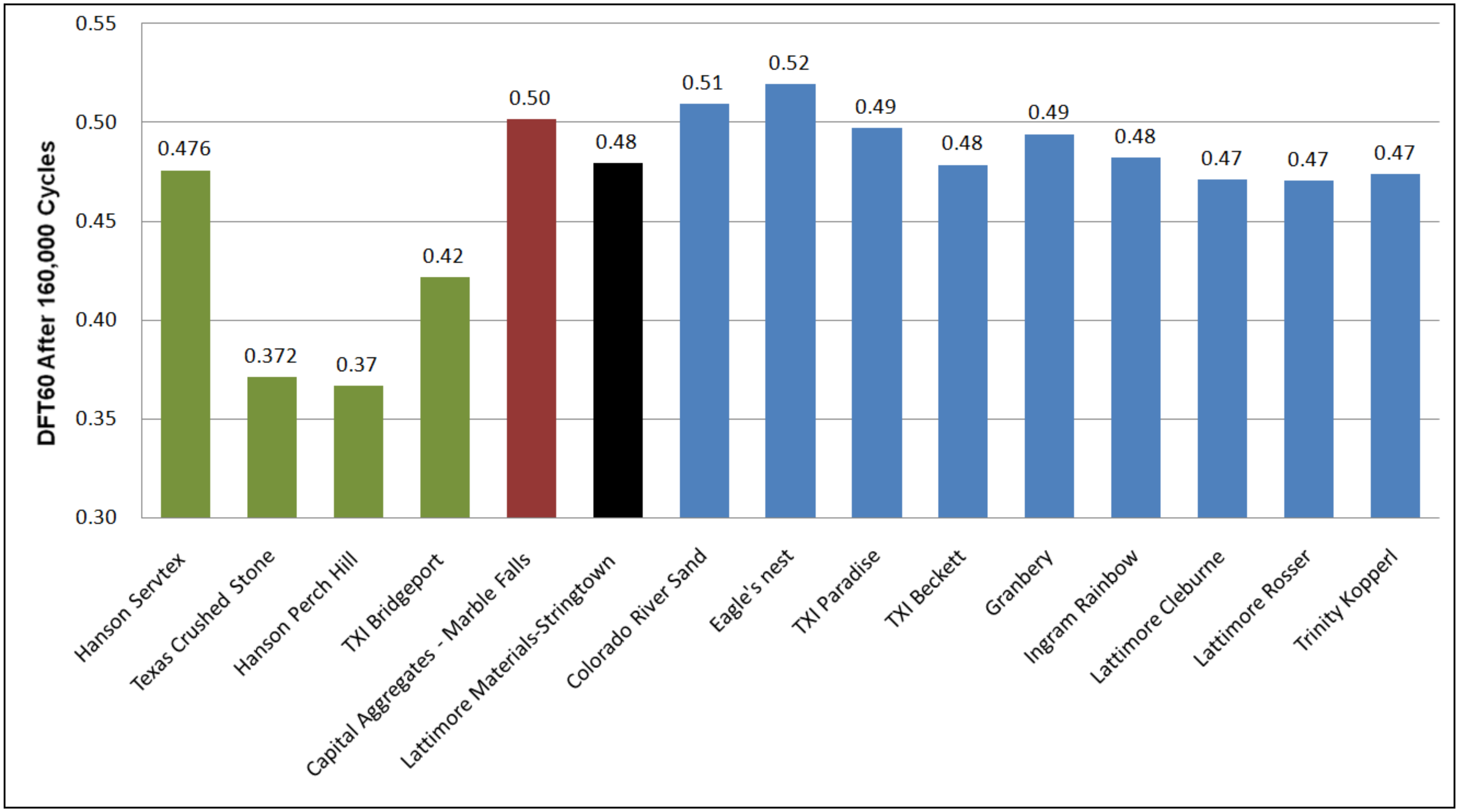

Figure 10.13: DFT60 Results at 160,000 Cycles for the Different Sands Tested

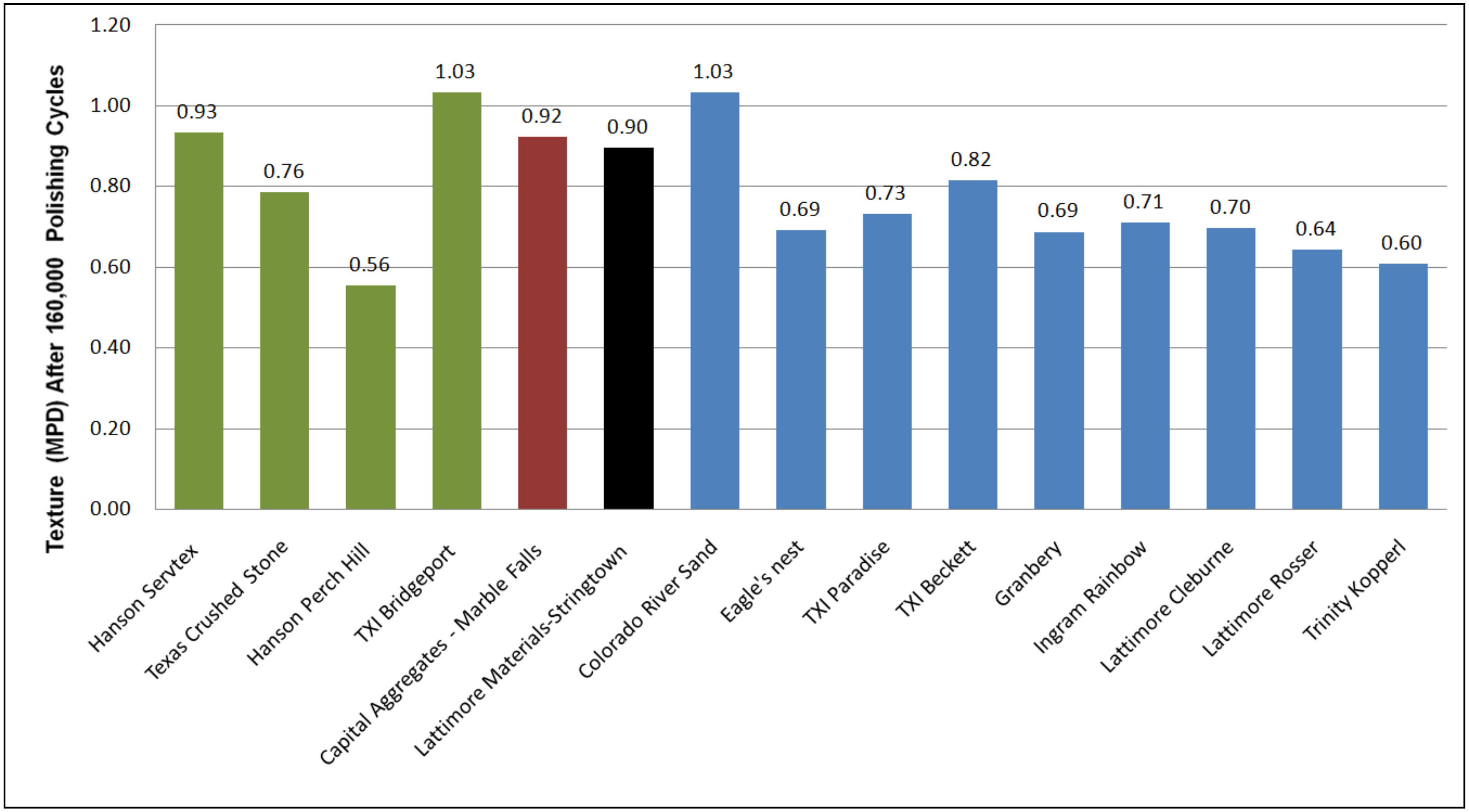

Figure 10.14: Texture Results at 160,000 Cycles for the Different Sands Tested

The values obtained from the CTM (MPD) and DFT (DFT20) were used to compute the international friction index (IFI). The IFI values computed were then used to calculate the equivalent skid numbers at 40mph using smooth tires (Figure 10.15) and ribbed tires (Figure 10.16). The formulas used to compute the skid numbers were presented and discussed in 3.3.4.

The values obtained using smooth tires shows that only the slab made with Hanson Perch Hill reached the trigger value of 20. The $SN(40)_{Smooth}$ values for some of the manufactured sands was higher than that of some of the siliceous sands. Those values were obtained because some of the manufactured sands had higher texture values, and $SN(40)_{Smooth}$ values are affected by micro-texture and macro-texture.

Ribbed tires are influenced by the micro-texture more than the macro-texture of a concrete pavement. The formula provided by ASTM E 1960 for ribbed tires takes that into account by reducing the effect of macro-texture on the computed skid value (refer to 3.3.4). Using such a formula resulted in skid numbers that better reflected the expected performance (except for Hanson Servtex). One of the limestone sands tested had values slightly higher than the trigger value of 30 after 160,000 polishing cycles.

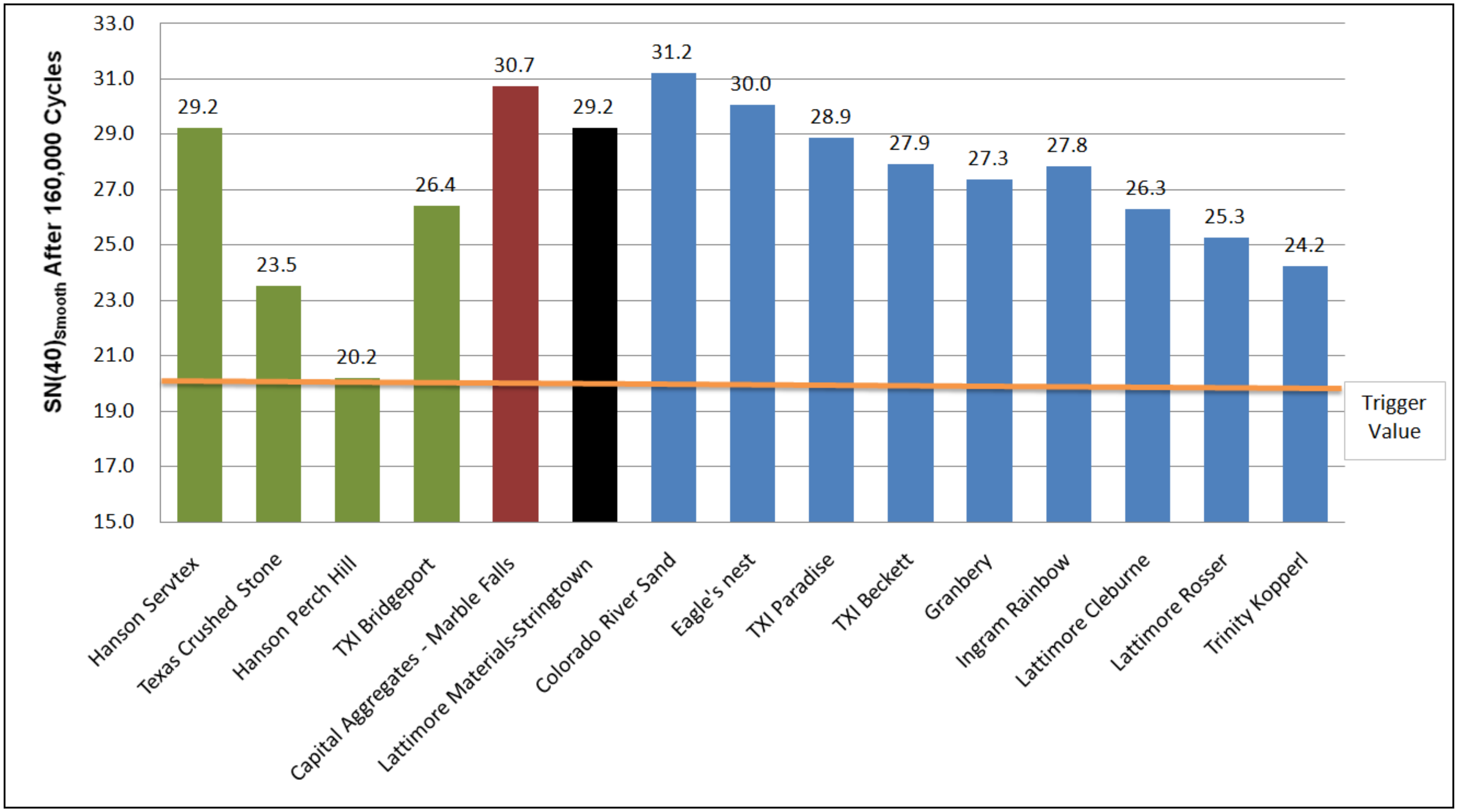

Figure 10.15: $SN(40)_{Smooth}$ Results at 160,000 Cycles for the Different Sands Tested

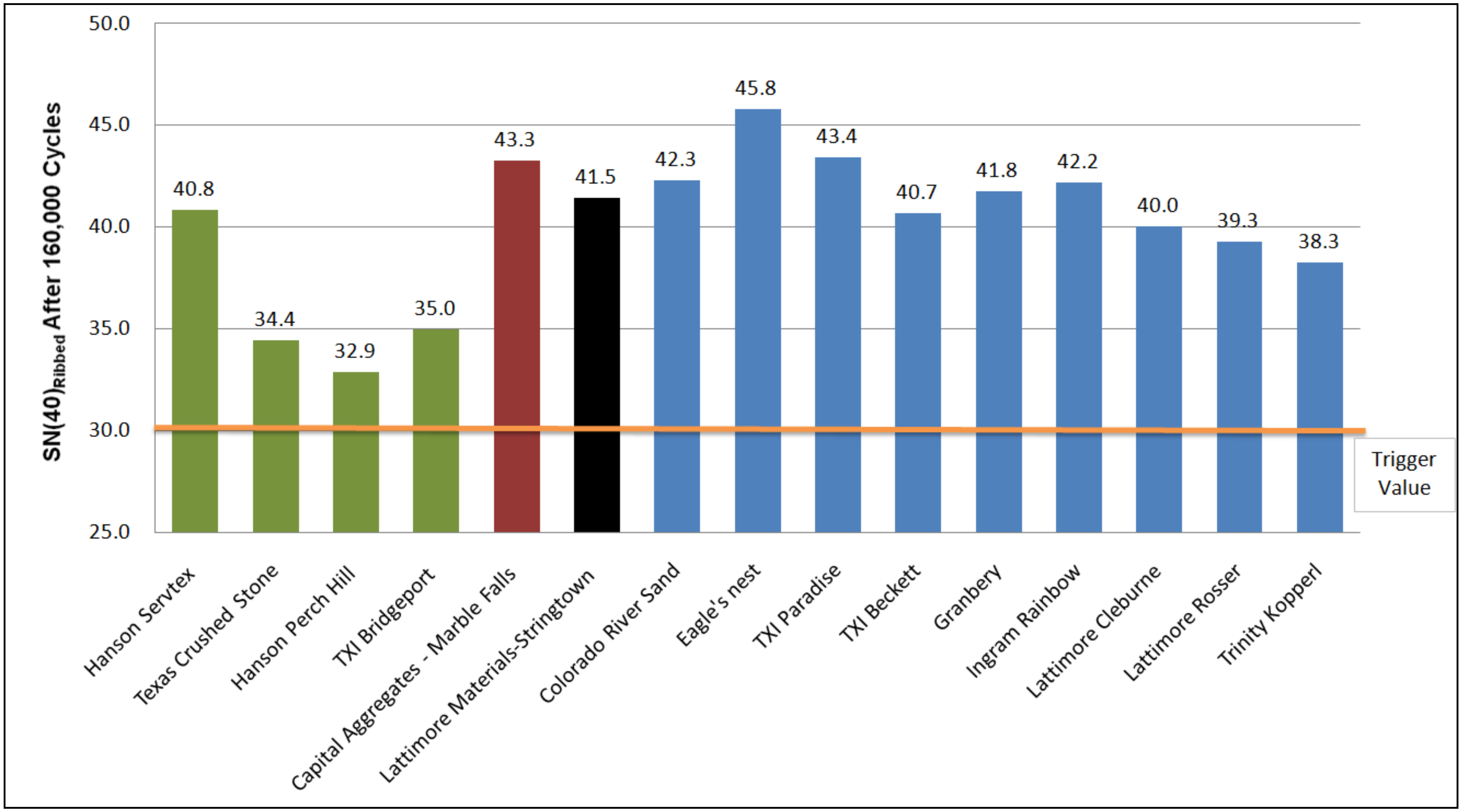

Figure 10.16: $SN(40)_{Ribbed}$ Results at 160,000 Cycles for the Different Sands Tested

10.2.3 Blended Sands

Fine aggregate that do not meet the AIR requirements are blended with sands that have higher AIR values to meet the specifications. The current TxDOT specifications require sands to meet an AIR value of 60%. Therefore, it was important to evaluate blends of sands that had AIR values lower than 60%. For this purpose two different sand combinations with AIR values of 20%, 40%, and 60% were tested. One contained TXI Paradise (siliceous) and TXI Bridgeport (limestone), and the other contained Trinity Kopperl (siliceous) and Hanson perch Hill (limestone). The mixture proportions used for making the concrete slabs were the same proportions presented in Table 10.1, but for those mixtures a 30% fly ash replacement was used. There were no indications from the literature reviewed that using fly ash might influence skid resistance; the reason fly ash was used for this test program was because the sponsor wanted the blended sand mixtures to represent concrete mixtures commonly used in PCC pavements. All PCC pavements currently used in Texas contain fly ash.

10.2.3.1 TXI Paradise/TXI Bridgeport Blends

The sand blends used to obtain an AIR of 20%, 40%, and 60% for the TXI Paradise and TXI Bridgeport combination are shown in Table 10.3. Note the proportions of sands shown in Table 10.3 are mass and not volume percentages.

TXI Bridgeport (%)	TXI Paradise (%)	Acid Insoluble Residue (%)	Lithology
0	100	74.4	Siliceous
20	80	60.0	Blended
47	54	40.3	Blended
74	26	20.4	Blended
100	0	1.3	Limestone

Table 10.3: AIR Values for TXI Paradise/TXI Bridgeport Combinations

The results for compressive strength at 7 and 28 days are shown in Figures 10.17 and 10.18. The 7-day compressive strengths of the blended sands were lower than that of the mixtures containing 100% siliceous or limestone aggregate. The lower strength was obtained because the blended sands contained 30% fly ash. The 28-day compressive strengths for the blended sands were similar to that of the concrete made with 100% siliceous and the concrete made with 100% limestone MFA.

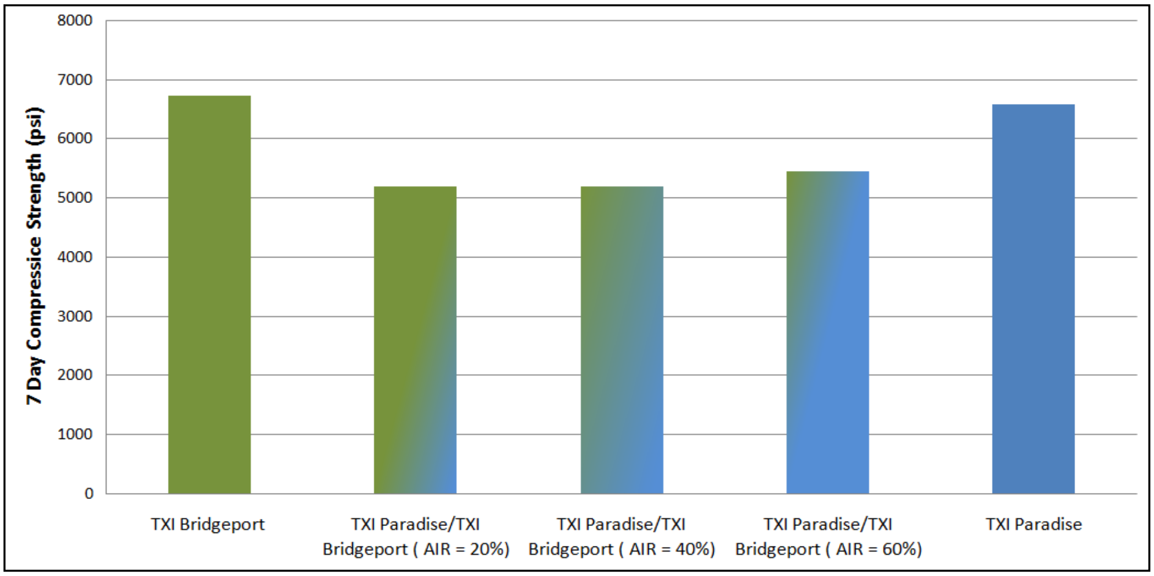

Figure 10.17: Compressive Strength of Concrete made with TXI Paradise/TXI Bridgeport Combinations after 7 days of Curing

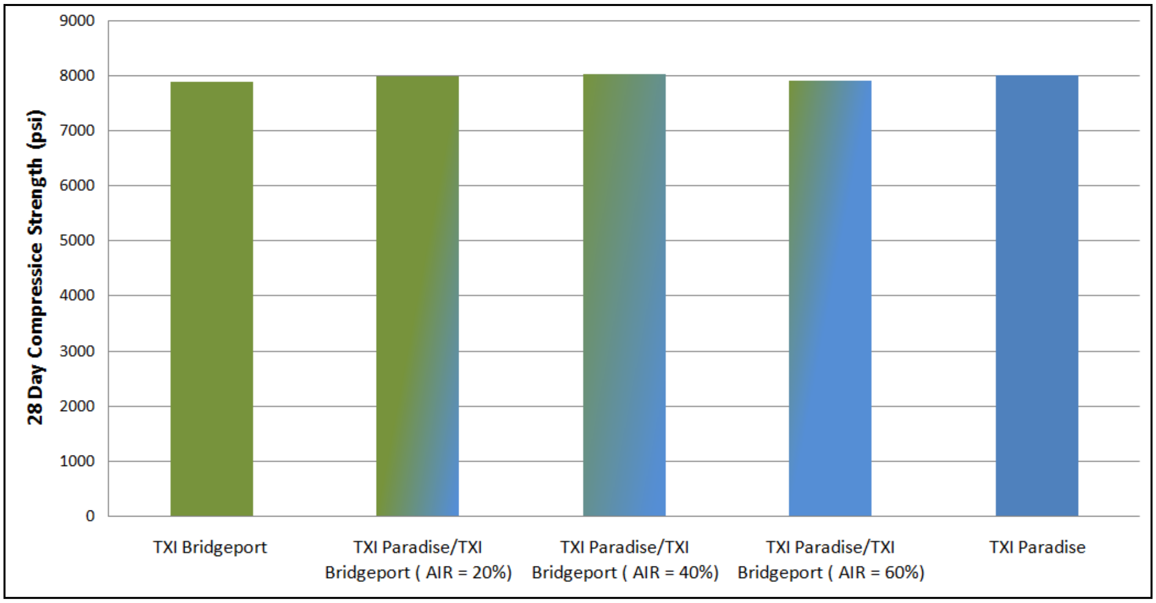

Figure 10.18: Compressive Strength of Concrete made with TXI Paradise/TXI Bridgeport Combinations after 28 days of Curing

The modulus of elasticity values at 28 days (Figure 10.19) for the blended sands was similar to that of the mixture containing 100% siliceous sand (TXI Paradise). The shrinkage values for the blended sand mixtures were not different from what was obtained when 100% siliceous or limestone aggregate was used (Figure 10.20). Blending sands does not have a large impact on modulus of elasticity or shrinkage.

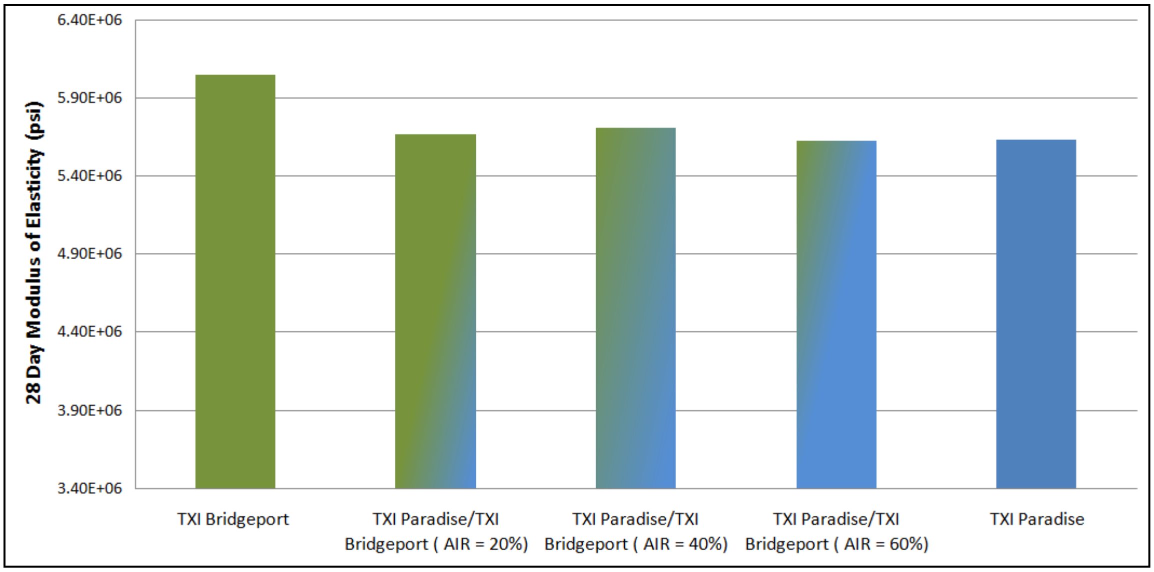

Figure 10.19: Modulus of Elasticity of Concrete made with TXI Paradise/TXI Bridgeport Combinations after 28 days of Curing

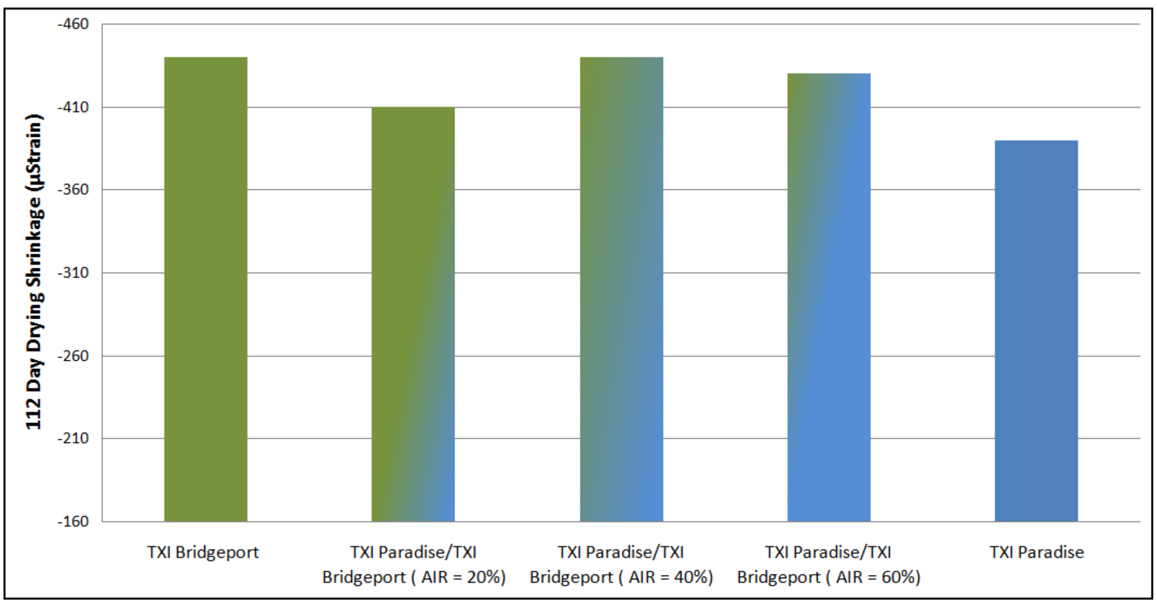

Figure 10.20: Drying Shrinkage of Concrete made with TXI Paradise/TXI Bridgeport Combinations

The DFT60 and texture results for the TXI Paradise and TXI Bridgeport blends are shown in Figures 10.21 and 10.22. The average difference in MPD between two slabs made from the same material was 0.141 while the difference in DFT60 values was 0.0191. The DFT60 values for the mixtures containing higher siliceous sand content (or higher AIR) were higher. The texture values were higher for the mixture containing no siliceous sand. The decrease in friction and texture followed trends similar to what was discussed in 10.2.2.

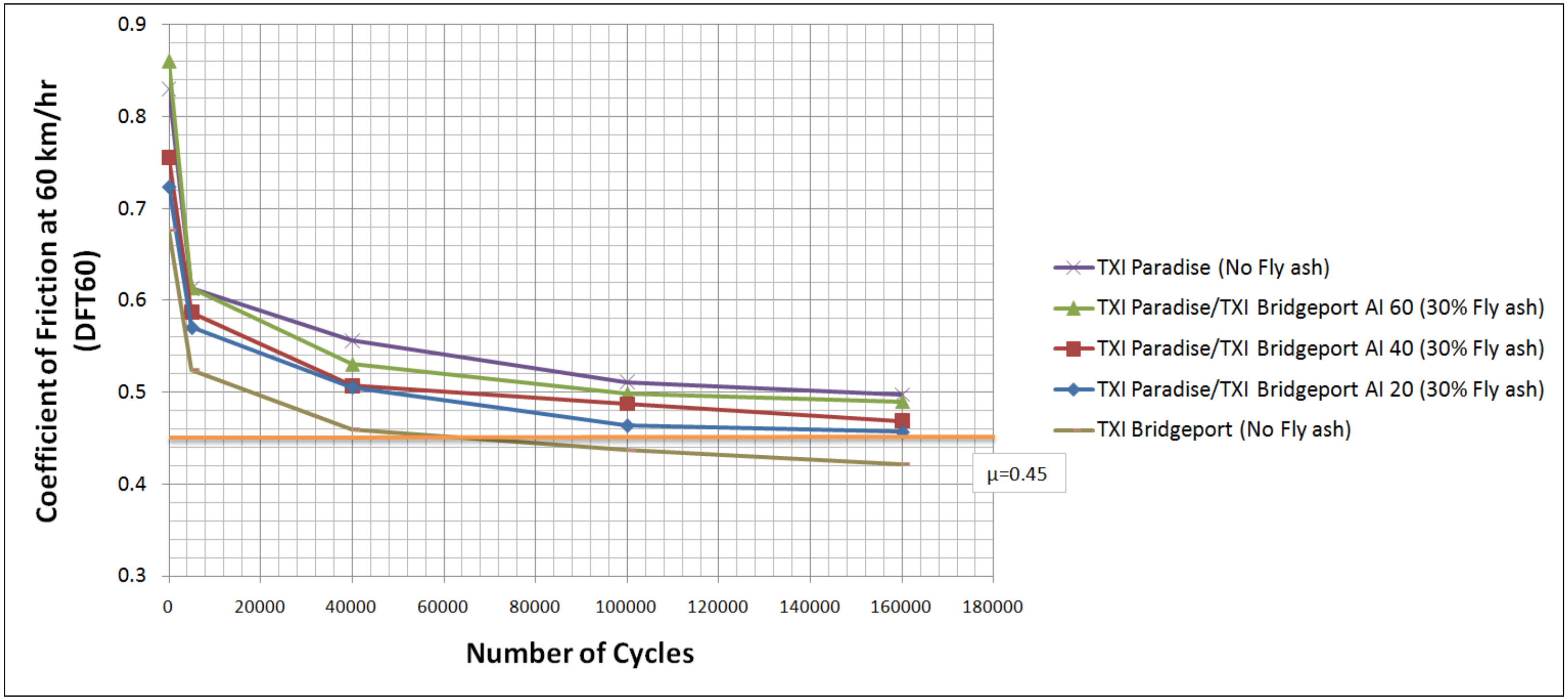

Figure 10.21: DFT60 Results for TXI Paradise/TXI Bridgeport Combinations

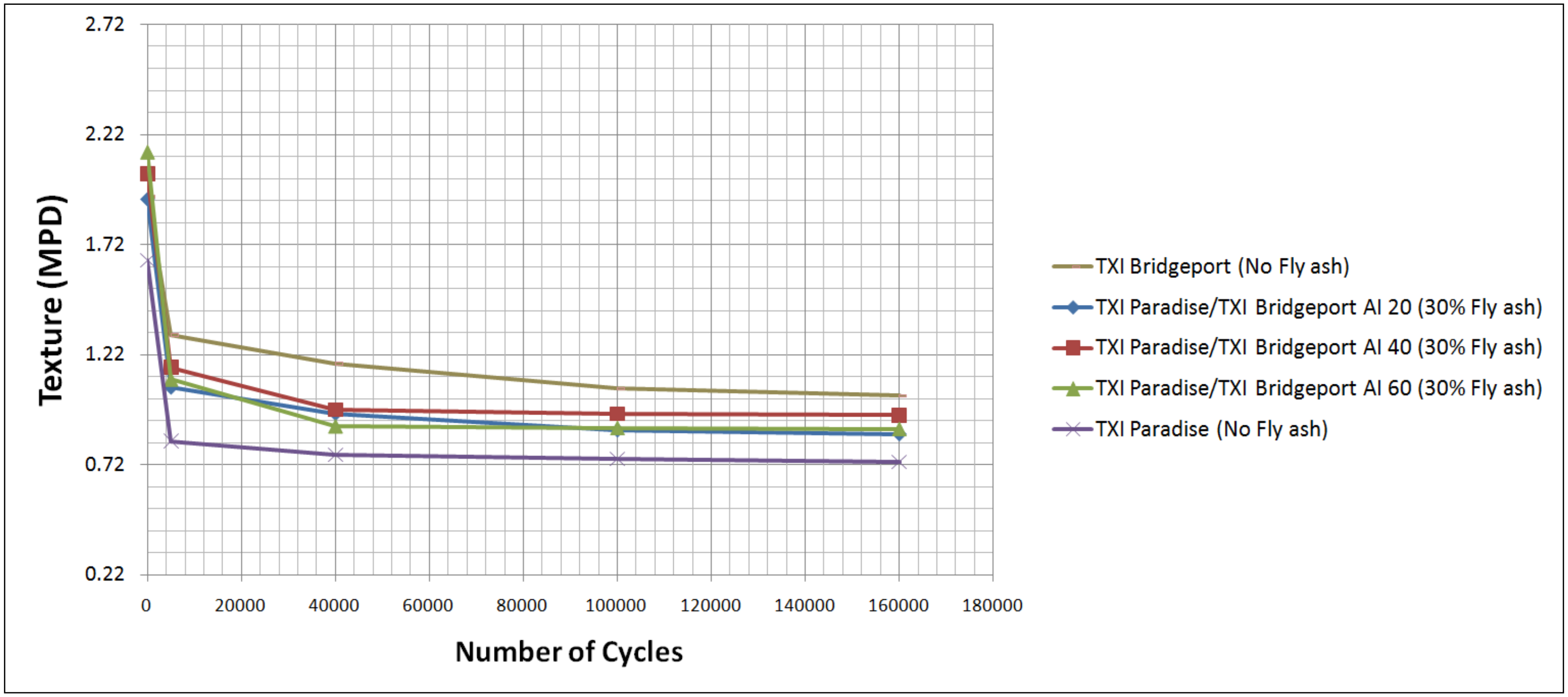

Figure 10.22: Texture Results for TXI Paradise/TXI Bridgeport Combinations

The DFT60 values at 160,000 cycles (Figure 10.23) increased as the siliceous content increased. The mixture with an AIR of 20% only had 26% siliceous sand but performed almost as well as the mixture with an AIR value of 40% which had a siliceous content of 54%. There was no significant difference between the mixture containing 100% siliceous sand and the mixture that had an AIR of 60%. All in all, adding a small quantity of siliceous sand had a large effect on skid performance. Note that the average difference in MPD between two slabs made from the same material was at 160,000 cycles was 0.111 while the difference in DFT60 values was 0.0057.

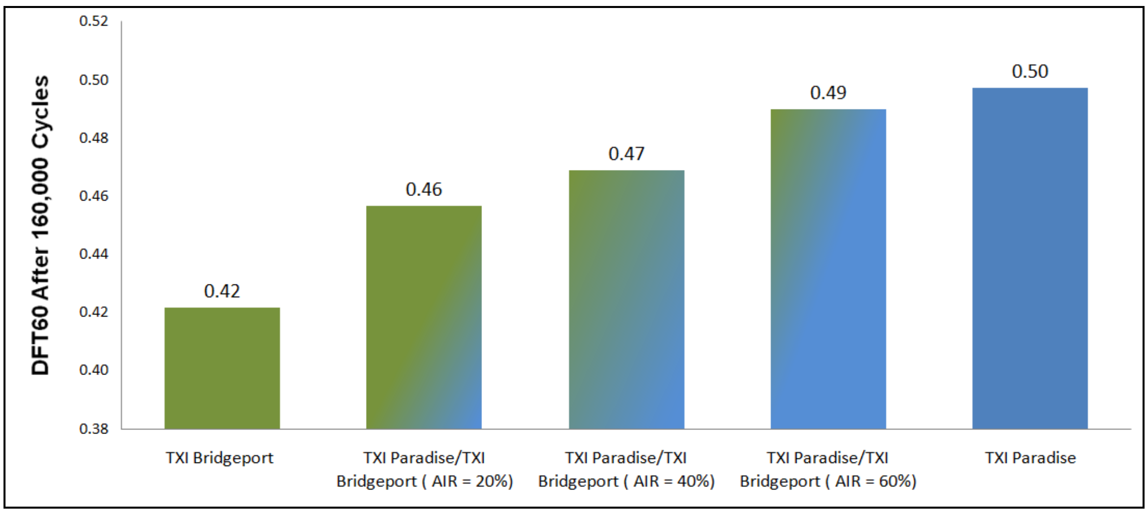

Figure 10.23: DFT60 Results at 160,000 Cycles for TXI Paradise/TXI Bridgeport Combinations

After 160,000 cycles, the texture value of the mixture containing 100% siliceous sand was lower than the texture obtained from all the other mixtures shown in Figure 10.24. The mixture containing 100% TXI Bridgeport had the highest texture after 160,000 polishing cycles.

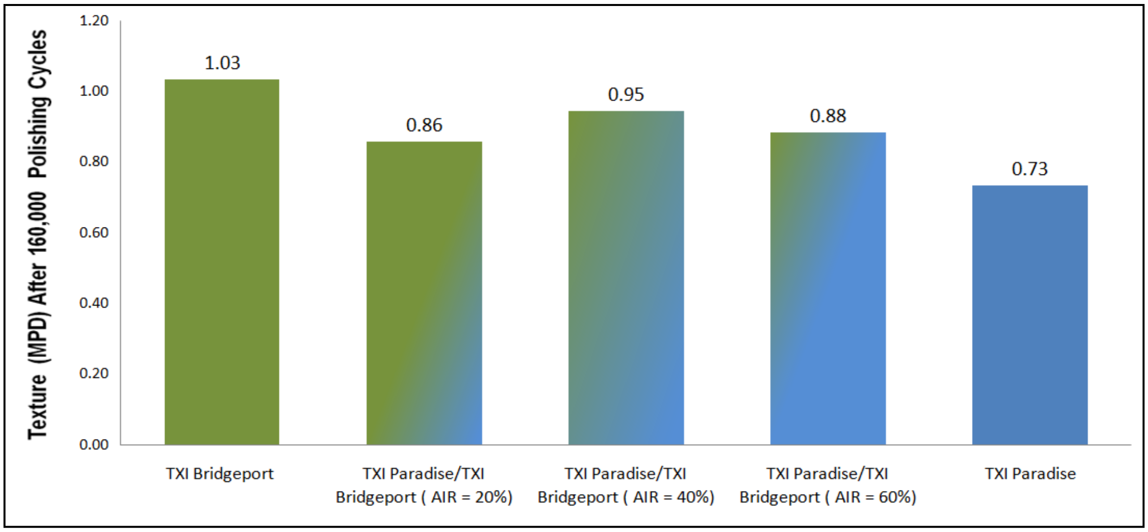

Figure 10.24: Texture Results at 160,000 Cycles for TXI Paradise/TXI Bridgeport Combinations

10.2.3.2 Trinity Kopperl/Hanson Perch Hill Blends

The sand blends used to obtain an AIR of 20%, 40%, and 60% for the Trinity Kopperl and Hanson Perch Hill combination is shown in Table 10.4.

Hanson Perch Hill (%)	Trinity Kopperl (%)	Acid Insoluble Residue (%)	Lithology
0	100	76.8	Siliceous
24	76	60.0	Blended
53	47	39.7	Blended
81	19	20.1	Blended
100	0	6.7	Limestone

Table 10.4: AIR Values for Trinity Kopperl/Hanson Perch Hill Combinations

The results for compressive strength at 7 and 28 days are shown in Figures 10.25 and 10.26. The 7-day compressive strength of the blended sands was lower than that of the mixtures containing 100% siliceous or limestone aggregate. The lower strength was obtained because the blended sands contained 30% fly ash. The 28-day compressive strength for the blended sands was similar to that of the concrete made with 100% siliceous and 100% limestone MFA.

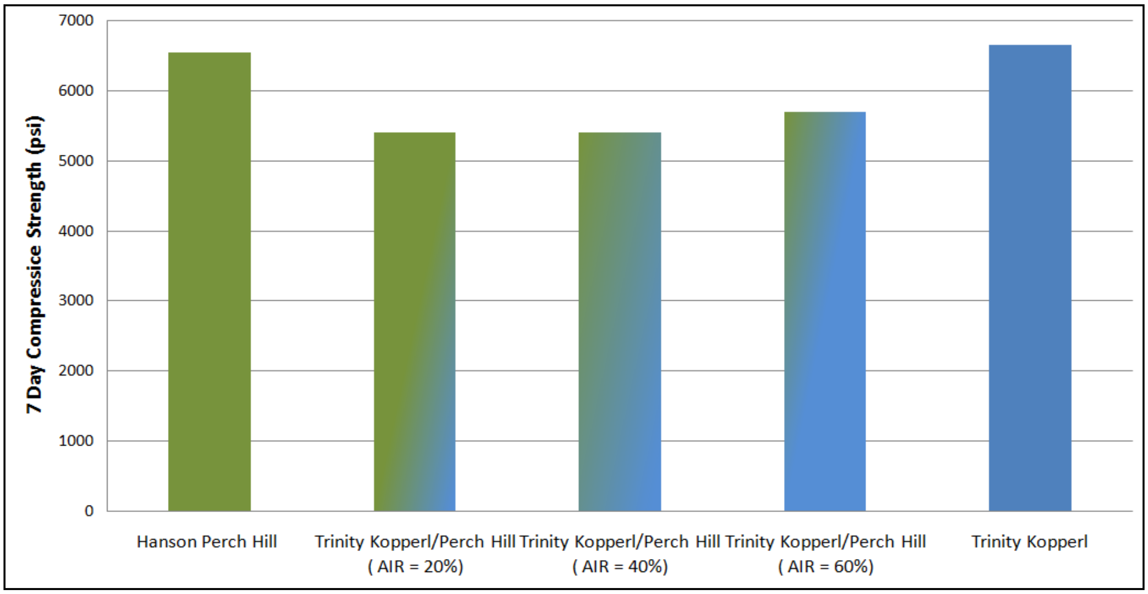

Figure 10.25: Compressive Strength of Concrete made with Trinity Kopperl/Hanson Perch Hill Combinations after 7 days of Curing

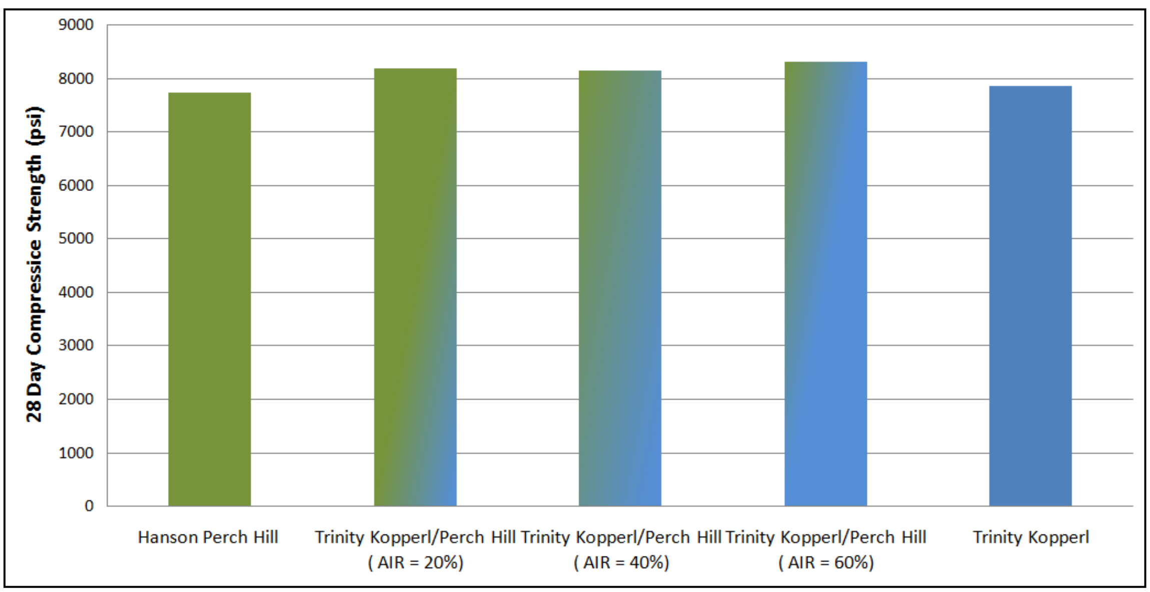

Figure 10.26: Compressive Strength of Concrete made with Trinity Kopperl/Hanson Perch Hill Combinations after 28 days of Curing

The modulus of elasticity values at 28 days (Figure 10.27) for the blended sands was similar to that of the mixture containing 100% limestone sand (Hanson Perch Hill). The shrinkage values for the blended sand mixtures were slightly higher than the shrinkage values obtained using 100% Trinity Kopperl sand (Figure 10.28). The relation between blended sands, modulus of elasticity, and shrinkage was not clear for both fine aggregate combinations tested.

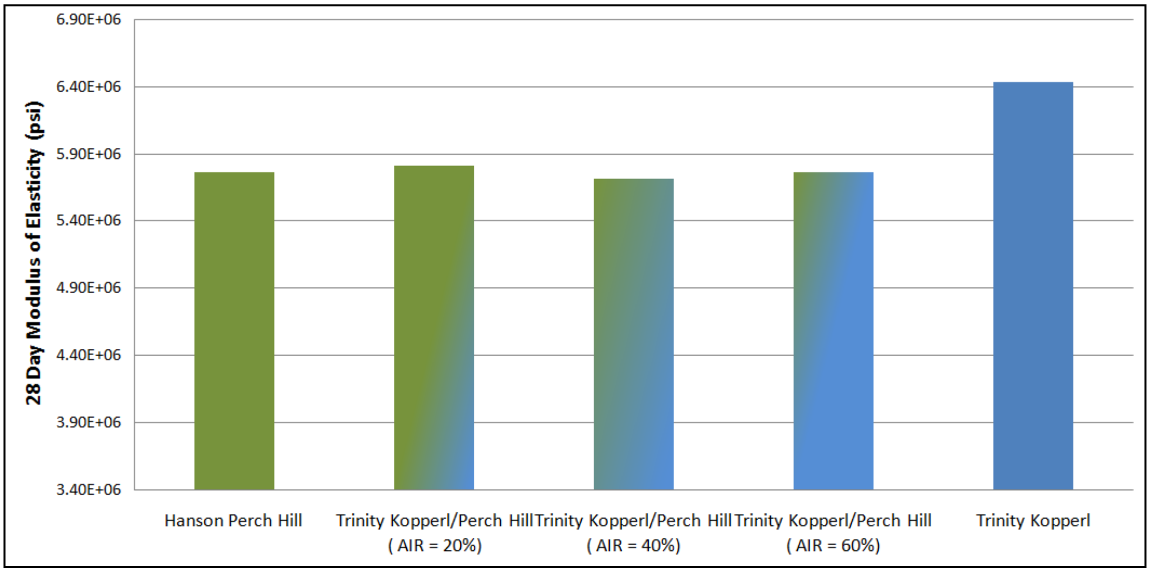

Figure 10.27: Modulus of Elasticity of Concrete made with Trinity Kopperl/Hanson Perch Hill Combinations after 28 days of Curing

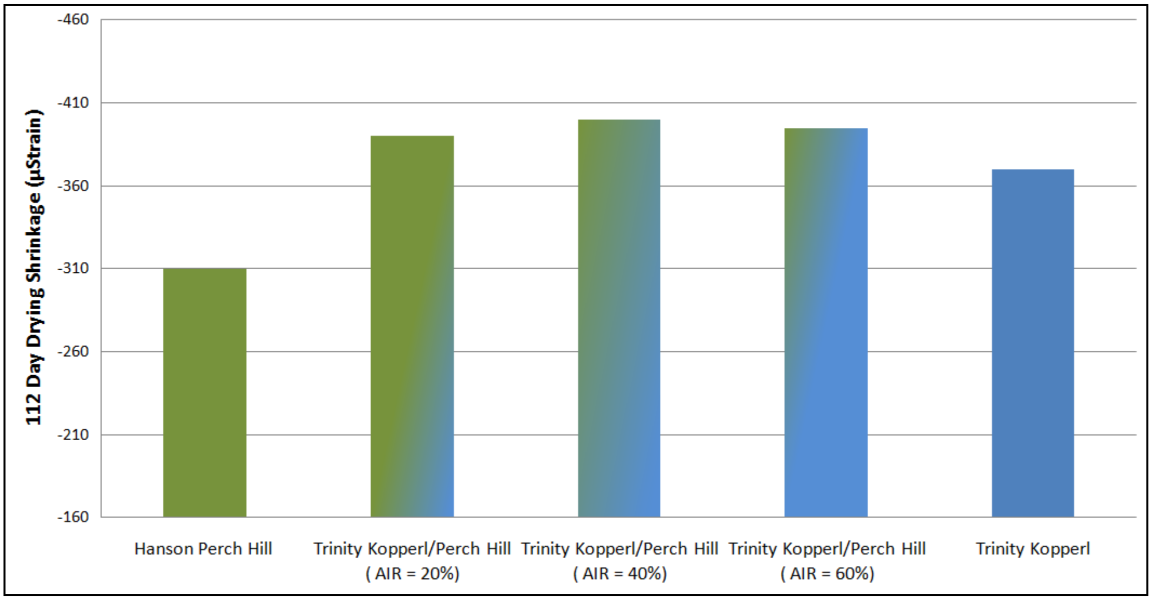

Figure 10.28: Drying Shrinkage of Concrete made with Trinity Kopperl/Hanson Perch Hill Combinations

The DFT60 and texture results for the Trinity Kopperl and Hanson Perch Hill blends are shown in Figures 10.29 and 10.30. The average difference in MPD between two slabs made from the same material was 0.097 while the difference in DFT60 values was 0.0176. The DFT60 values for the mixtures containing any amount of siliceous sand content were significantly higher than the mixture made with 100% limestone MFA (Hanson Perch Hill). The highest texture values were obtained for the mixture with an AIR of 20%.

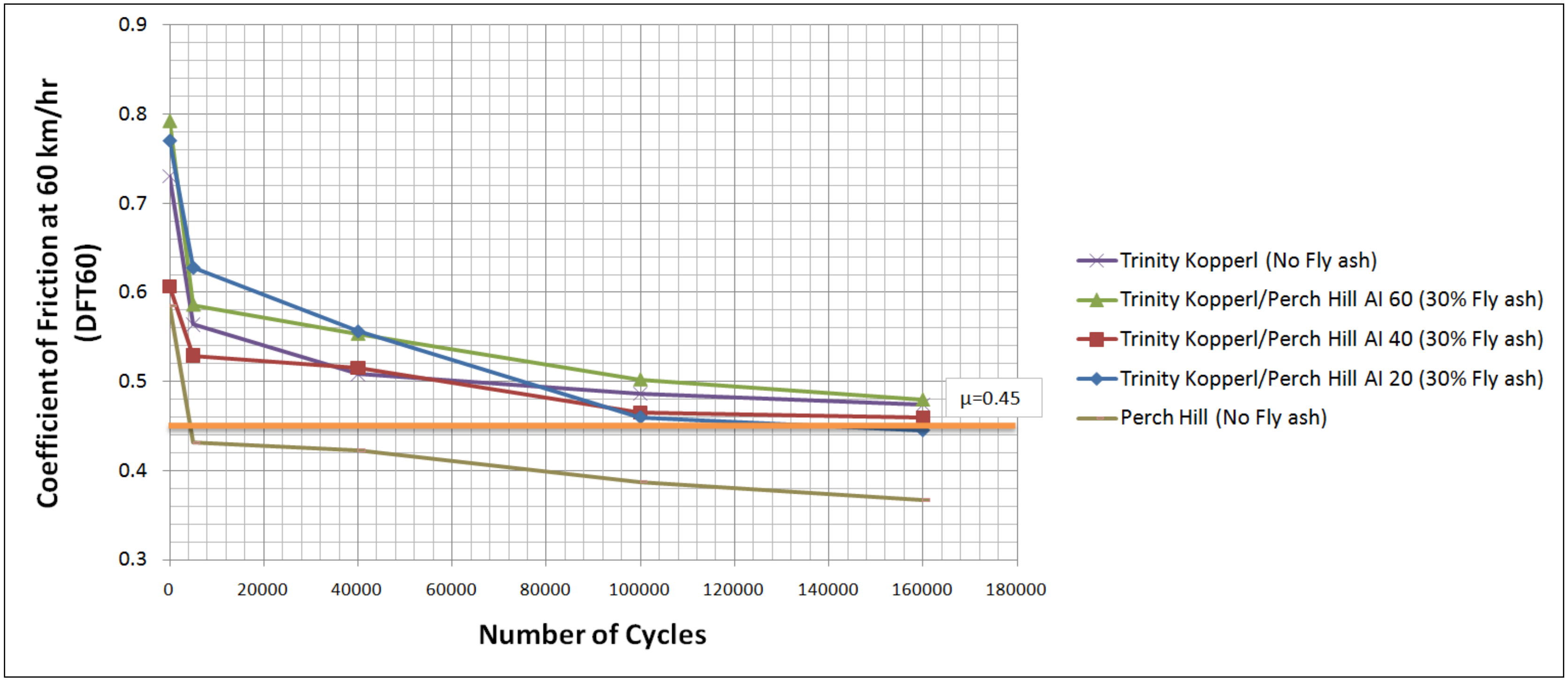

Figure 10.29: DFT60 Results for Trinity Kopperl/Hanson Perch Hill Combinations

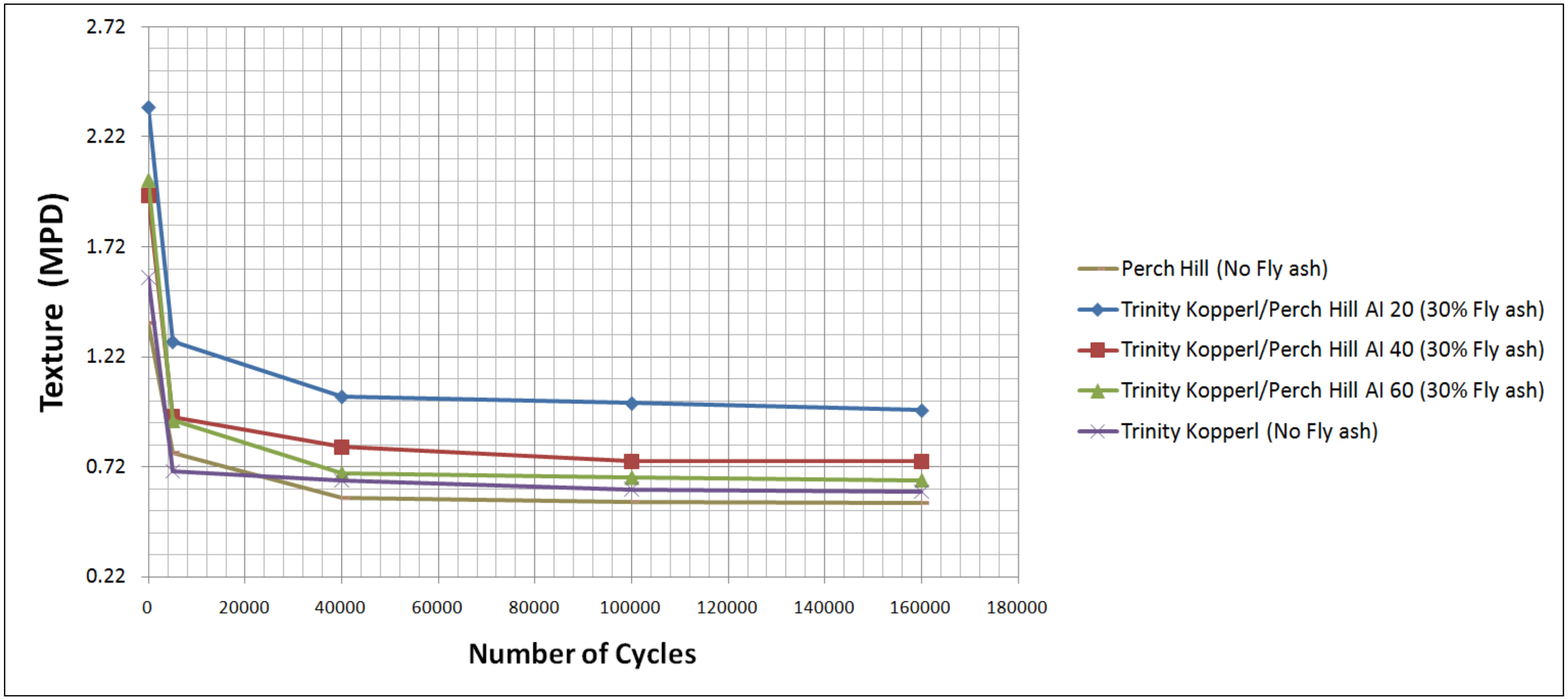

Figure 10.30: Texture Results for Trinity Kopperl/Hanson Perch Hill Combinations

The DFT60 values at 160,000 cycles (Figure 10.31) increased for the blended sands as the siliceous content increased. By only adding 19% Trinity Kopperl (AIR of 20%), the DFT60 value was increased from 0.37 to 0.45. This shows that adding a small quantity of siliceous sand to concrete mixtures has a significant effect on friction. Such results are similar to the findings obtained by Balmer and Colley (1966) which indicated that adding 25% siliceous sand would result in satisfactory skid performance. Note that the average difference in MPD between two slabs made from the same material was at 160,000 cycles was 0.066 while the difference in DFT60 value was 0.0056.

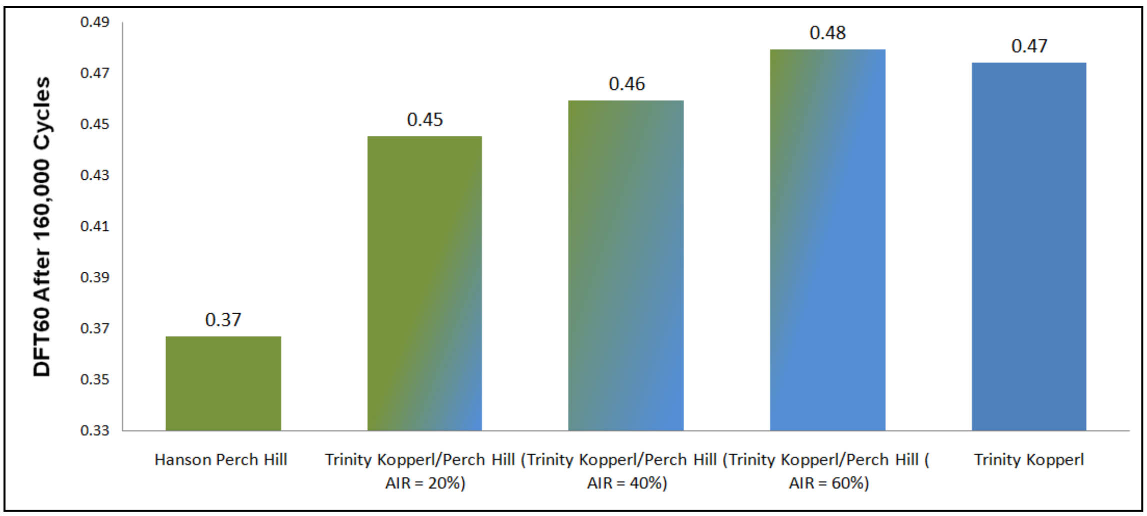

Figure 10.31: DFT60 Results at 160,000 Cycles for Trinity Kopperl/Hanson Perch Hill Combinations

There was no trend in the change of texture when different blends of Trinity Kopperl and Hanson Perch Hill were used (Figure 10.32). The highest texture at 160,000 cycles was obtained with a mixture having an AIR of 20%; lower texture values were obtained when less or more siliceous sand was used.

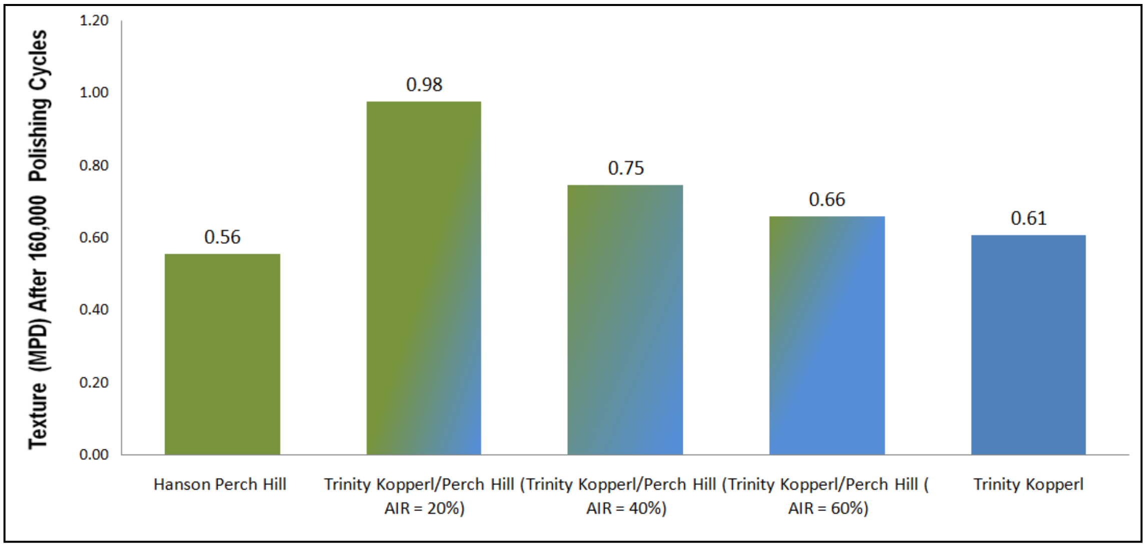

Figure 10.32: Texture Results at 160,000 Cycles for Trinity Kopperl/Hanson Perch Hill Combinations

10.3 Evaluating the Effect of Mixture Proportions on Texture and Friction of PCC

10.3.1 Mixture Proportions

In section 10.2, all mixtures were evaluated using the same mixture proportions. In this section the effect of changing mixtures proportions was investigated using limestone MFA. The seven different mixture proportions used are shown in Table 10.5.

	Materials (Volume %)			
	Cement	Water	Fine Aggregate	Coarse Aggregate
Baseline Mixture	10.73	14.20	27.06	46.01
W/C = 0.39	11.19	13.74	27.06	46.01
W/C = 0.45	10.31	14.62	27.06	46.01
S/A = 0.3	10.73	14.20	21.92	51.15
S/A = 0.44	10.73	14.20	32.15	40.92
5.25-Sack Mixture	9.30	12.30	28.29	48.11
6.75-Sack Mixture	12.00	15.88	25.97	44.16

Table 10.5: Mixture Proportions used for Evaluating the Effect of Proportioning on Skid

10.3.2 Test Results

Figures 10.33 and 10.34 show the friction and texture results for the same sand mixed at three different water-to-cement ratios. The initial DFT60 values were not equal. As the numbers of cycles increased the DFT60 value seemed to converge to the same value for the three different mixtures. Changing the mixture design by varying the water-to-cement ratio between 0.39 and 0.45 does not seem to affect the DFT60 values after 160,000 polishing cycles. The texture values were different for two out of the three mixtures. The highest texture was obtained with the mixture that had the highest water-to-cement ratio.

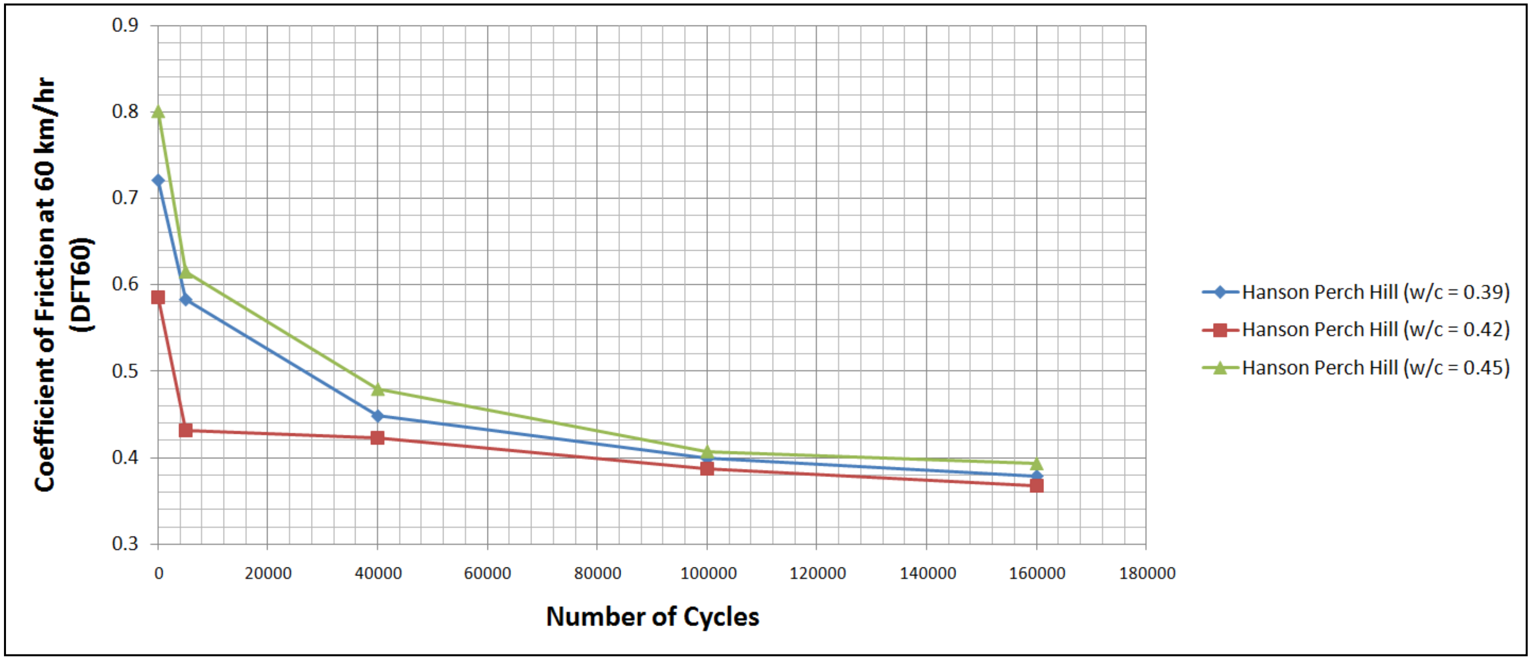

Figure 10.33: DFT60 Results for Mixtures Containing Hanson Perch Hill at three different w/c ratios

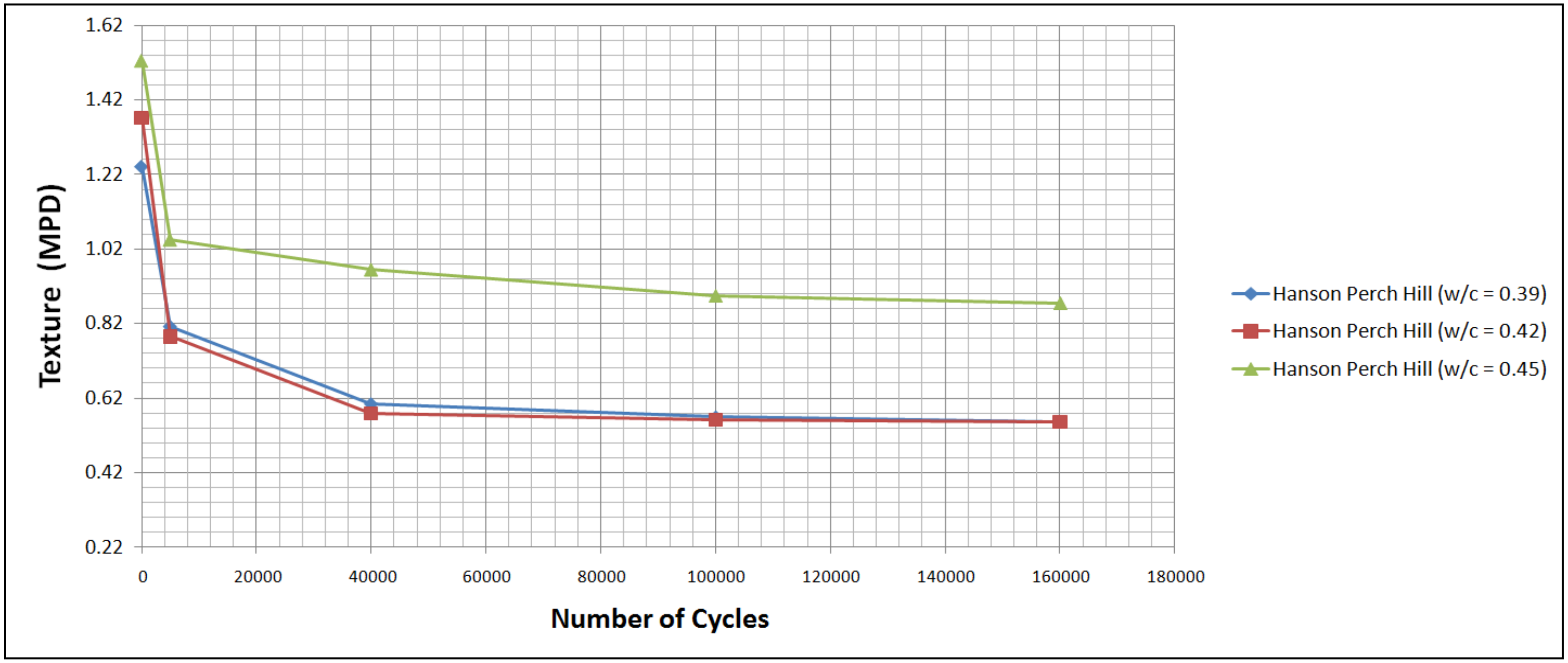

Figure 10.34: Texture Results for Mixtures Containing Hanson Perch Hill at three different w/c ratios

Figures 10.35 and 10.36 show the friction and texture results for the same sand mixed at three different sand-to-aggregate ratios. Changing the sand-to-aggregate ratio from 0.30 to 0.44 had no effect on the DFT60 value at 160,000 cycles. The DFT60 value seemed to have converged at 100,000 cycles, even though the starting values were not equal. As for texture, the mixture that had the lowest sand-to-aggregate ratio had the lowest texture; this might be due to the fact that that mixture had less sand which resulted in a concrete texture containing less poorly shaped particles.

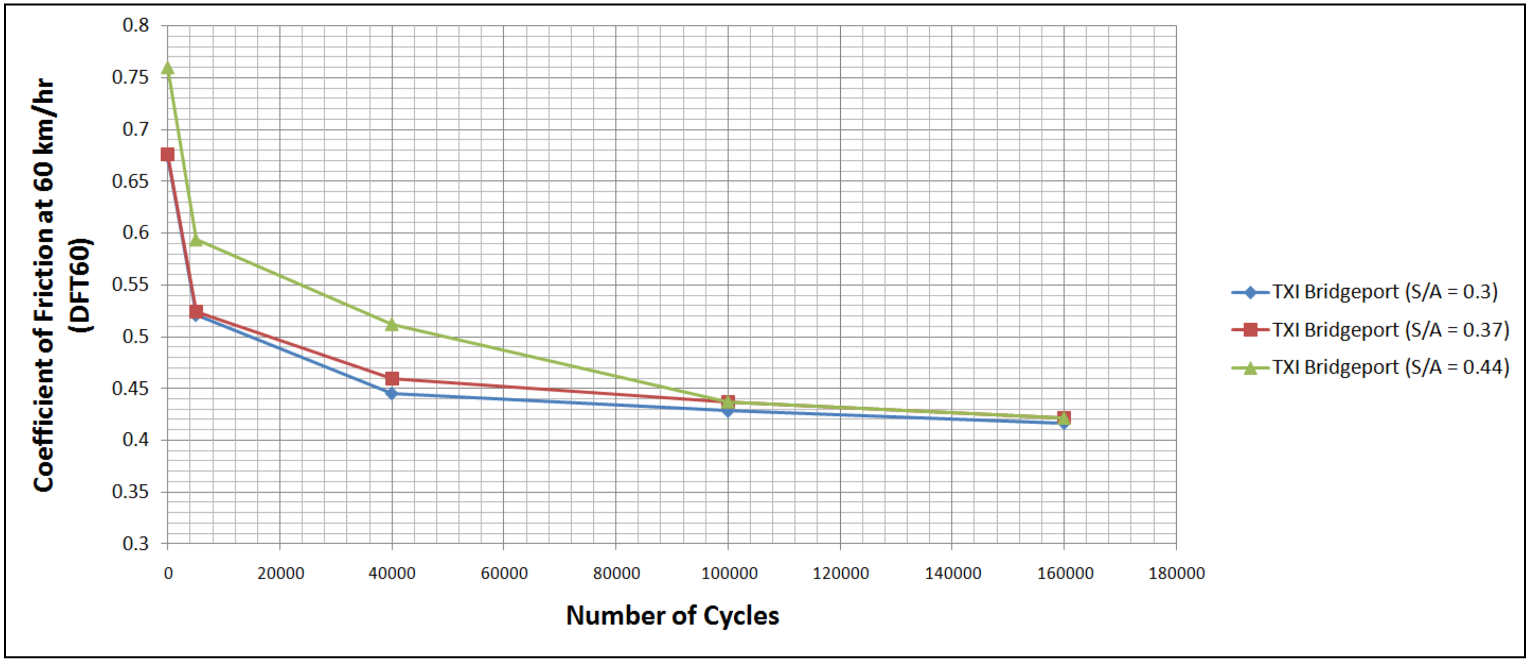

Figure 10.35: DFT60 Results for Mixtures Containing TXI Bridgeport at three different S/A ratios

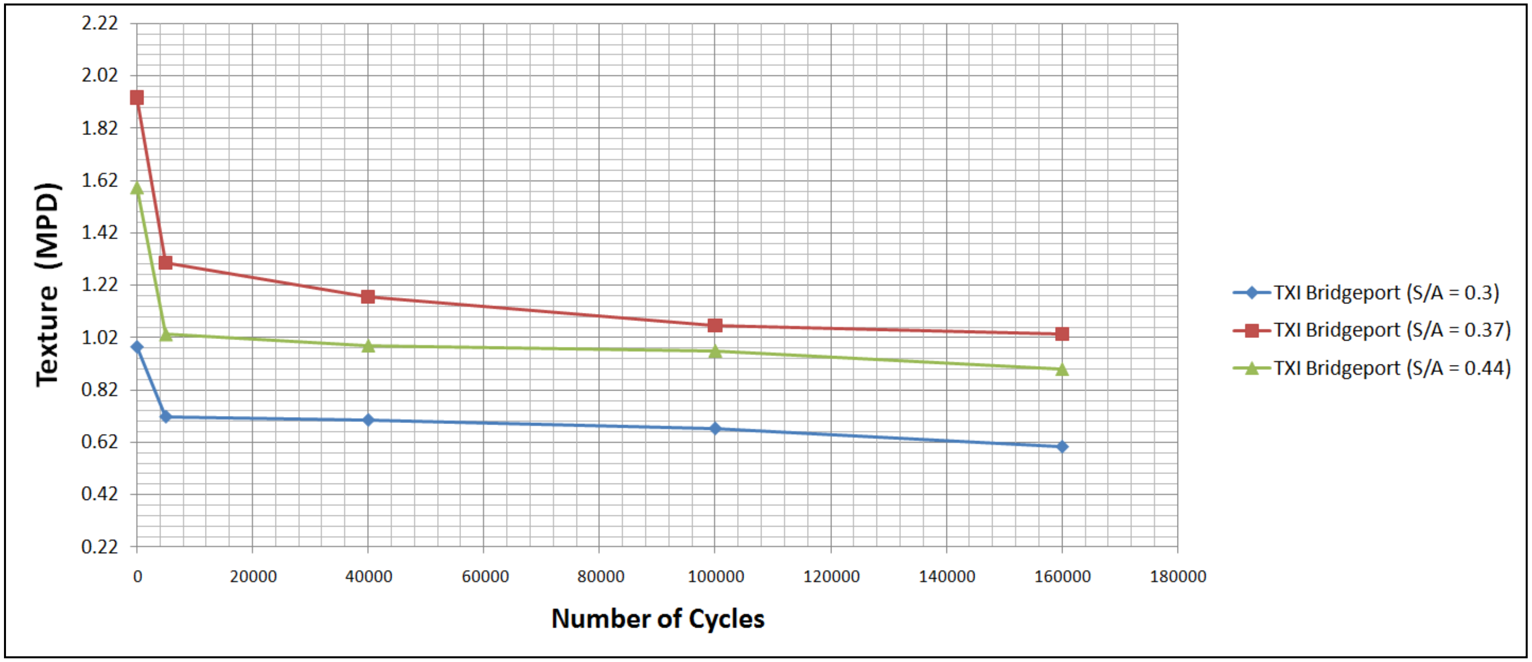

Figure 10.36: Texture Results for Mixtures Containing TXI Bridgeport at three different S/A ratios

Figures 10.37 and 10.38 show the friction and texture results for the same sand mixed at with different cement content. Increasing or decreasing cement content did not improve the DFT60 value of mixtures made with TXI Bridgeport. The DFT60 values measured converged after 160,000 polishing cycles. Results obtained in Figure 10.37 show that the mixtures with the highest cement content had the lowest initial texture and the lowest texture at 160,000 cycles. This might have occurred due to a change in fine aggregate content; as the cement content changed the paste content had to also be changed to maintain a constant water-to-cement ratio. This caused the total aggregate ratio to either decrease or increase when the cement content was changed (Table 10.5). As the fine aggregate content decreased, less texture was created because the mixtures had less poorly shaped fine aggregate and were not as harsh (had higher paste content).

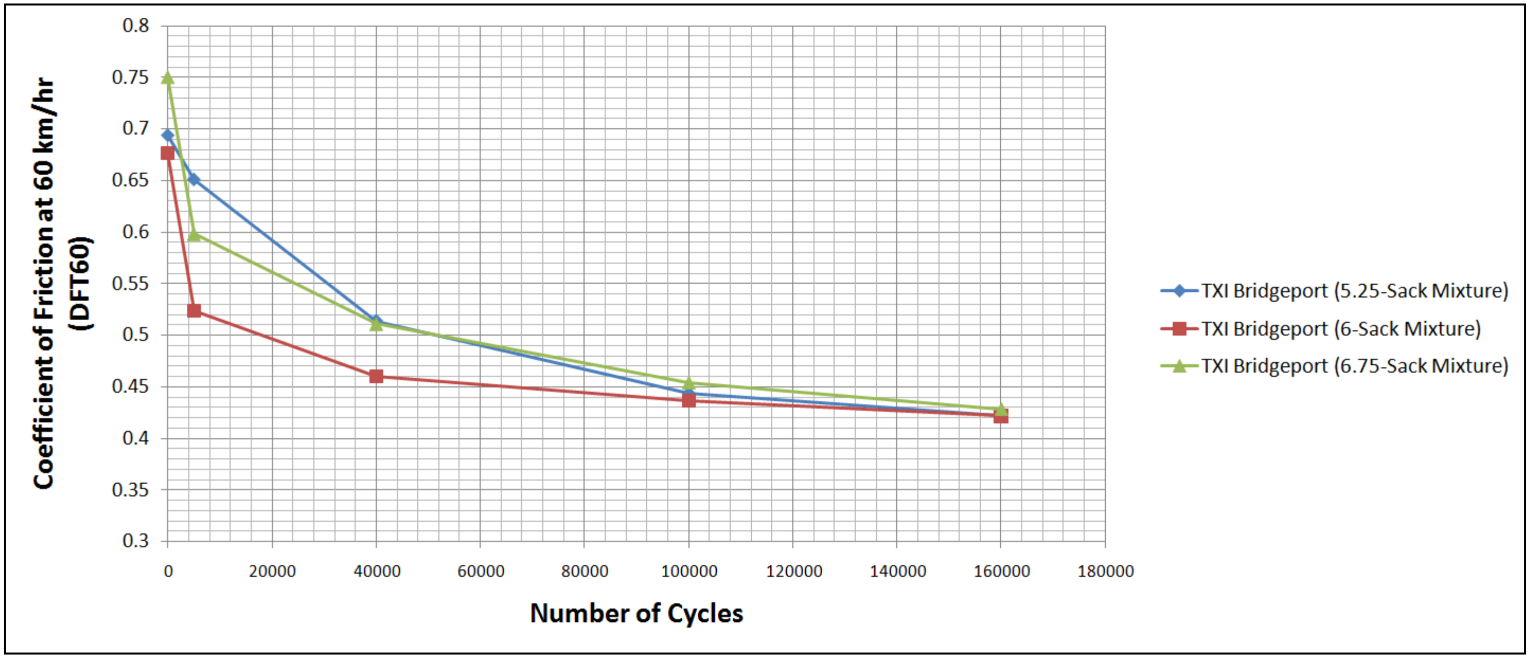

Figure 10.37: DFT60 Results for Mixtures Containing TXI Bridgeport with different Cement Content

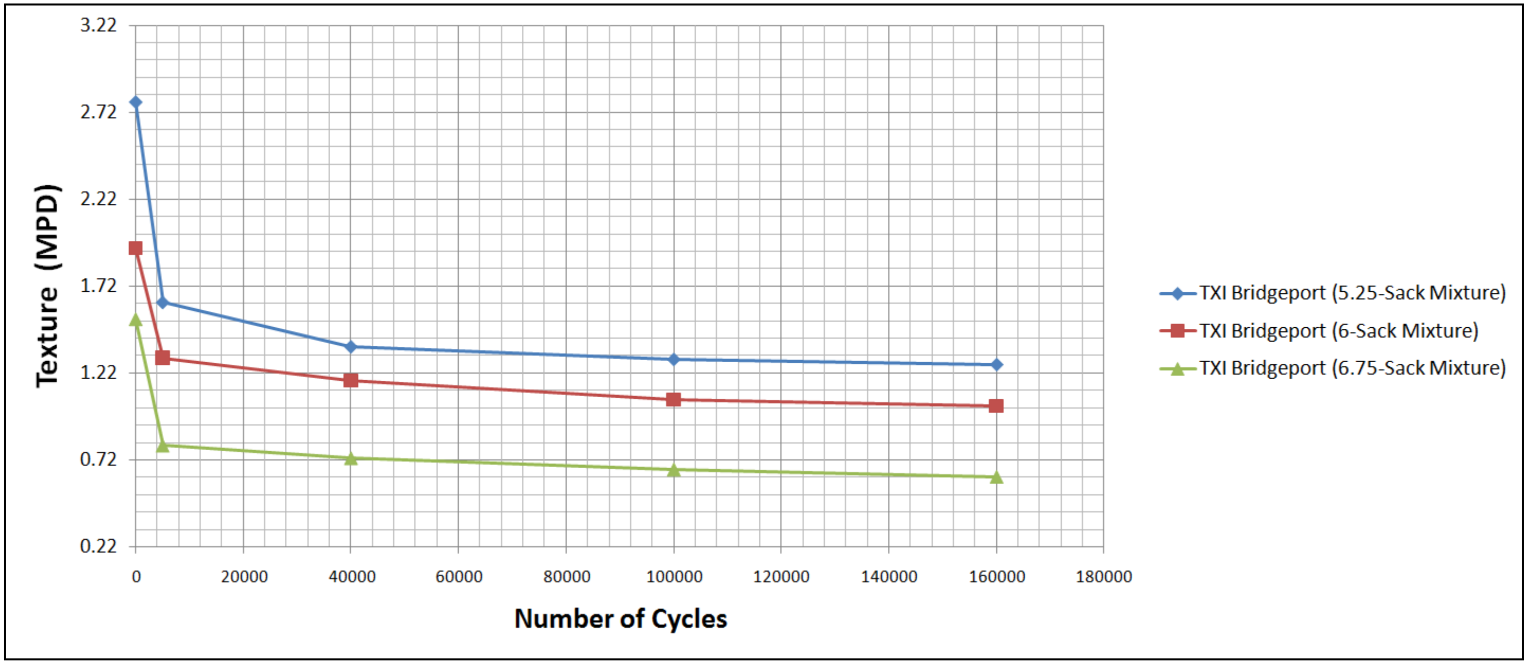

Figure 10.38: Texture Results for Mixtures Containing TXI Bridgeport with different Cement Content

10.4 Conclusions

The main goal of the concrete testing performed in this research project aimed at evaluating the skid resistance of concrete made with manufactured sands. While evaluating skid resistance of concrete made with MFA, it was also important to ensure that other concrete properties such as compressive strength, shrinkage, and modulus of elasticity were not negatively affected by the use of MFA. Based on the results obtained, it was found that:

- The use of manufactured sand in concrete does not lead to a reduction in compressive strength.
- Except for the dolomite sand tested, the modulus of elasticity obtained using manufactured sands was higher to that of concrete made with siliceous sand.
- The shrinkage values obtained with all carbonate aggregates was comparable to the shrinkage values obtained using siliceous sands. Concrete made with Lattimore Stringtown had higher shrinkage values than concrete made from all the other sands.
- The texture values (MPD) measured using the CTM did not correlate well with the expected performance of fine aggregates. Concrete made with soft manufactured limestone sands had MPD values that were sometimes higher than the MPD values obtained with concrete made with hard siliceous sands. The MPD value is thus not a good tool for evaluating the polish resistance of fine aggregates.
- The DFT60 values correlated well with the expected skid performance of fine aggregates in concrete. Except for Hanson Servtex, the values obtained after 160,000 cycles for the siliceous sands was higher than the values obtained for the limestone sands.

- The computed skid values obtained using the ribbed tire formula resulted in values that better represented the expected skid performance.
- The dolomite sand tested performed better than the limestone sands.
- Blending a small quantity of siliceous sand with soft limestone manufactured sands considerably increased the DFT60 value after 160,000 polishing cycles.
- Changing mixture proportions might have an effect on macro-texture (MPD values) but it did not have any effect on the DFT60 values after 160,000 TWPD cycles when limestone manufactured fine aggregates were used. Thus, the performance of limestone manufactured fine aggregates cannot be improved by changing mixture proportions.

Chapter 11: Analysis of Skid Data

The concrete testing discussed in Chapter 10 evaluated the polish resistance of 56 slabs made with 21 different fine aggregate and fine aggregate blends. The results showed that the polish resistance of concrete is mainly influenced by the type or blend of sands used. The results from Chapters 8 and 10 showed that friction values measured using the DFT could better evaluate the polish resistance of fine aggregates. In this chapter, the results obtained from the concrete skid tests in Chapter 10 were compared to the aggregate test results presented in Chapter 4. The goal in this chapter was to find an aggregate test that could predict skid performance. A formula that relates DFT60 and the skid number (SN) using a ribbed tire to micro-Deval percent loss was computed. Recommendations for an alternative method of evaluating and blending fine aggregates for pavement concrete is also presented in this chapter. This method aims at better quantifying the hardness of aggregates through their resistance to abrasion and crushing rather than their resistance to acid.

11.1 Finding a Correlation Between Aggregate and Concrete Properties

Figure 11.1 shows a plot that compares friction and texture values after 160,000 cycles. The results show that there was no correlation between the texture and friction values obtained. Since the DFT60 values better correlated with the expected skid performance of fine aggregates, and since there is no correlation between texture measured using the CTM and DFT measurements, texture values were not used for any aggregate and concrete correlation.

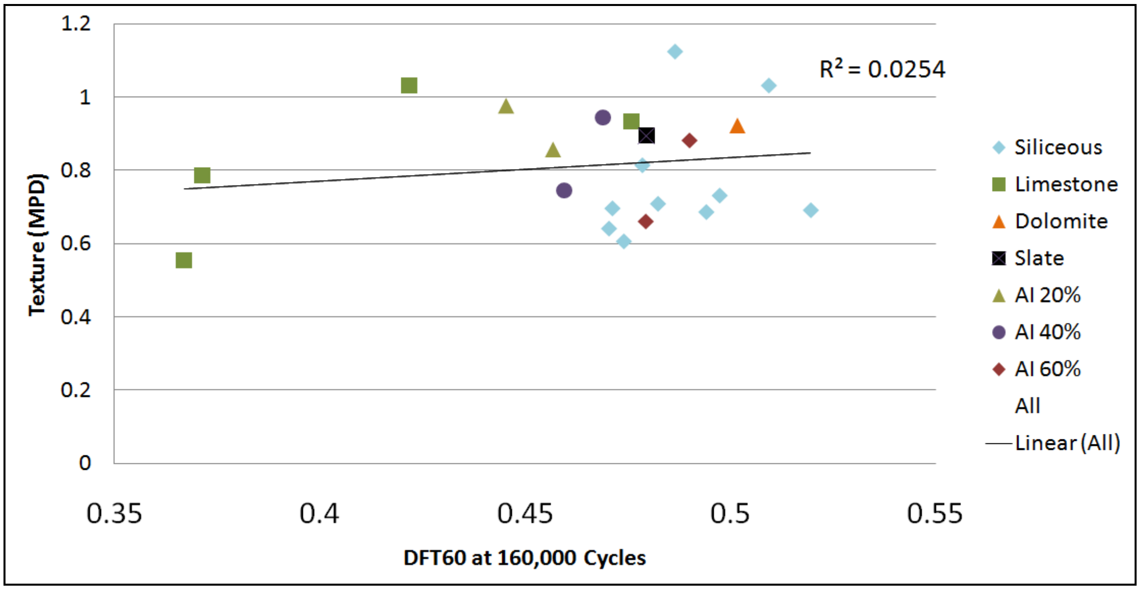

Figure 11.1: MPD vs. DFT60 after 160,000 cycles

Figure 11.2 shows is a plot of the DFT60 values and the acid insoluble residue values obtained for all mixtures. For the siliceous sands, slate sand, and blended sands, a decrease in AIR seemed to lead to a decrease in DFT60 values. On the other hand, there seemed to be no correlation between the AIR values and DFT60 values obtained for the carbonate aggregates (limestone and dolomite). The AIR did not differentiate between the performances of the different carbonate aggregates because all the carbonate fine aggregates completely dissolved in acid.

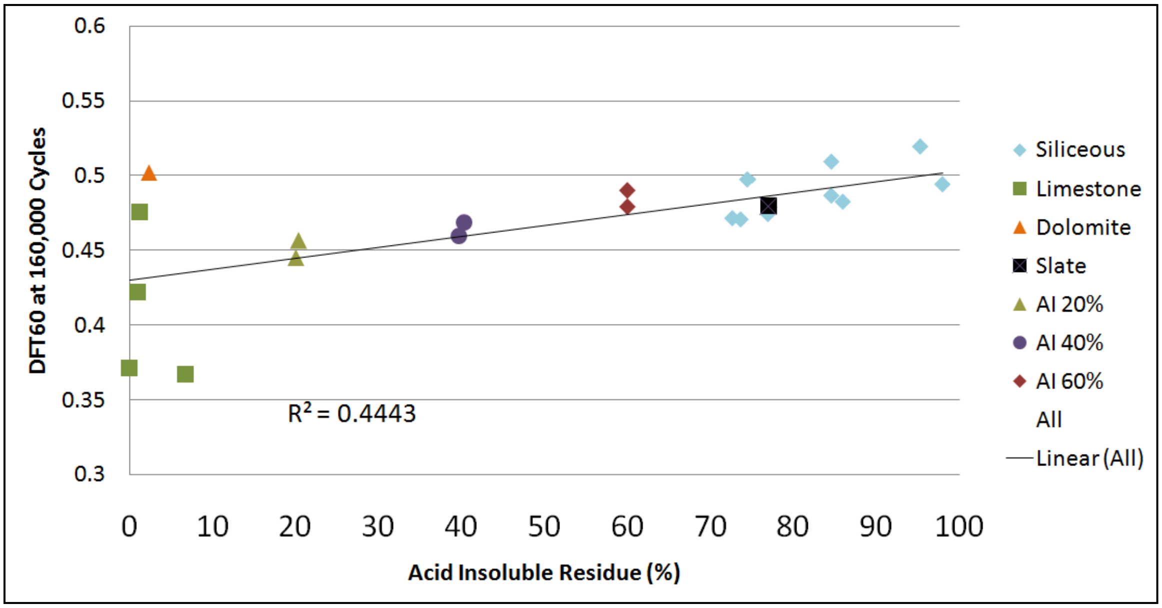

Figure 11.2: DFT60 vs. AIR

A better correlation was obtained using the micro-Deval test (Figure 11.3). Except for one limestone sand, all fine aggregates that had high micro-Deval percent loss performed poorly. The increase in micro-Deval percent loss for the blended sands was also associated with a decrease in the DFT60 value. The relationship between DFT60 and micro-Deval was not linear for all the sands tested.

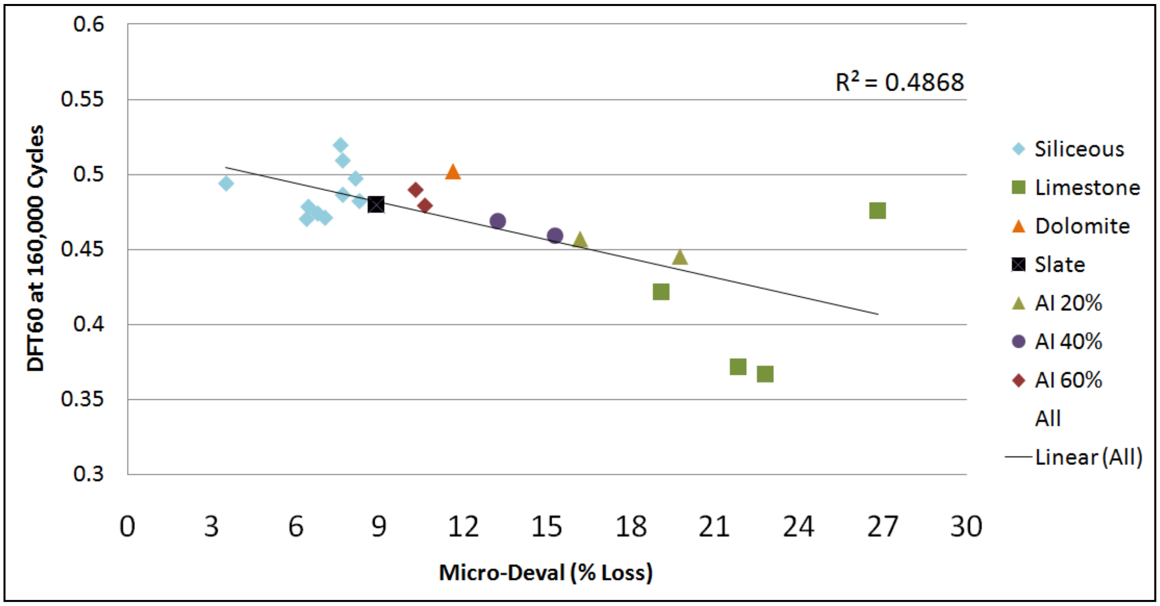

Figure 11.3: DFT60 vs. Micro-Deval

In Figure 11.4, the concrete performance is compared to the absorption capacity of aggregates. Although the DFT60 values correlate well with the absorption values, absorption would not be a good aggregate performance test because the method described in ASTM C 128 for determining the absorption capacity is subjective and not repeatable. Unless a better method of testing absorption was used, absorption determined by ASTM C 128 would not be a good measure of aggregate durability.

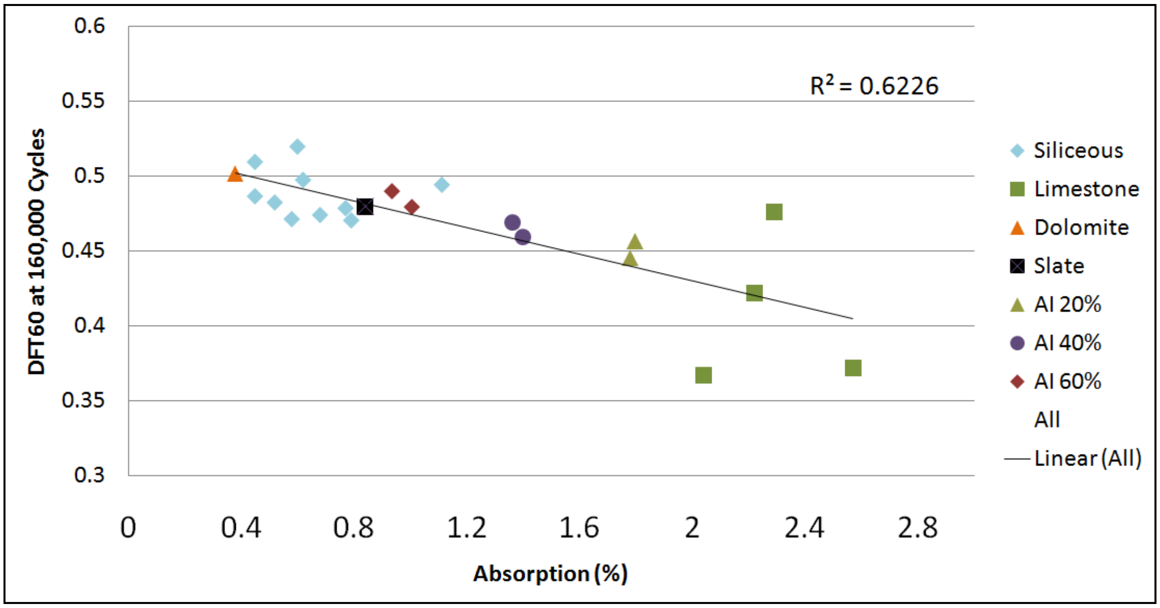

Figure 11.4: DFT60 vs. Absorption

11.2 Estimating DFT60 and SN(40)$_{\text{RIBBED}}$ Using the Micro-Deval Percent Loss Value

The AIR test used by TxDOT has insured that no skid resistance problems would be encountered when aggregates were blended to meet an AIR limit of 60%. The AIR test however is not a good tool for evaluating the hardness of aggregates; it is a chemical test that evaluates the carbonate content of aggregates which is believed to relate to hardness (it is a surrogate test). The obtained results from the laboratory concrete tests during this project and even the results obtained by Balmer and Colley (1966) showed that not all carbonate aggregates have the same laboratory skid performance. The micro-Deval test is a mechanical test that evaluates resistance of fine aggregates to abrasion and crushing. This is why the micro-Deval would be a more suitable test for evaluating the polish resistance of fine aggregates. Figure 11.5 shows that the micro-Deval correlates well with the AIR test. The only fine aggregate that performs well in micro-Deval but fails AIR is the dolomitic aggregate (Capital Marble Falls). That same aggregate had a DFT60 values after 160,000 TWPD cycles comparable to the values obtained with siliceous sands. Dolomites are known to be harder carbonate aggregates, and the reason they fail AIR is because they are carbonates.

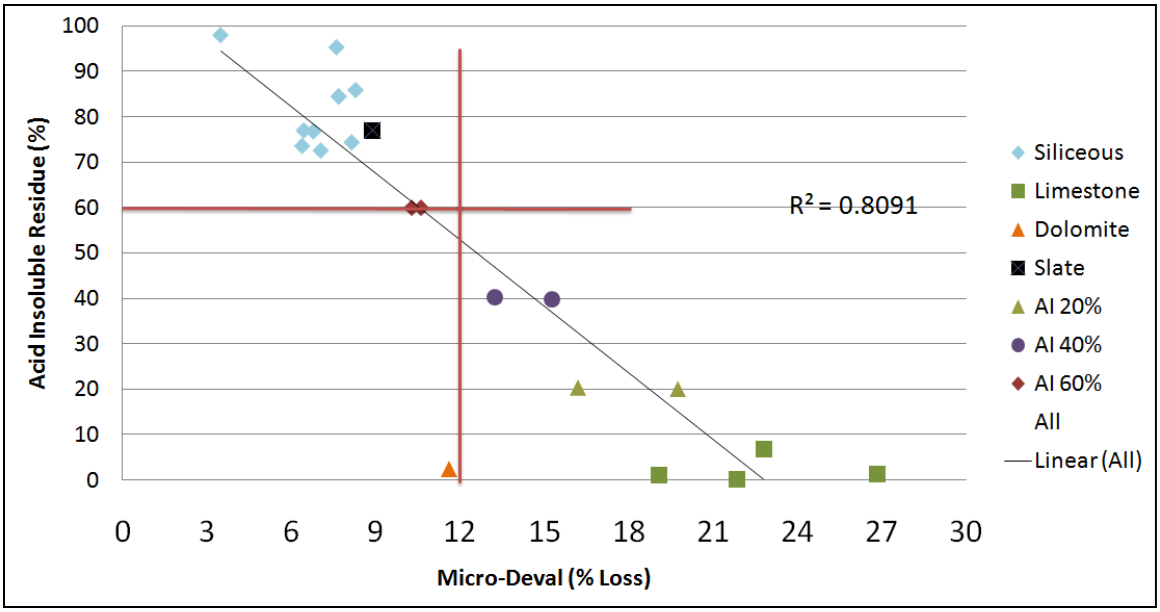

Figure 11.5: AIR vs. Micro-Deval

In Figure 11.3, the relationship between DFT60 and micro-Deval was observed to be nonlinear. Also, Hanson Servtex was the only aggregate that did not seem to follow the trend between increase in DFT60 values and decrease in micro-Deval percent loss. If the results presented in Figure 11.3 are plotted again without Hanson Servtex and if a polynomial function of the third degree is used as a trend-line instead of the a linear function, better correlation between DFT60 and micro-Deval values could be obtained (Figure 11.6). The function obtained from this correlation could be used to estimate the DFT60 value after 160,000 TWPD cycles.

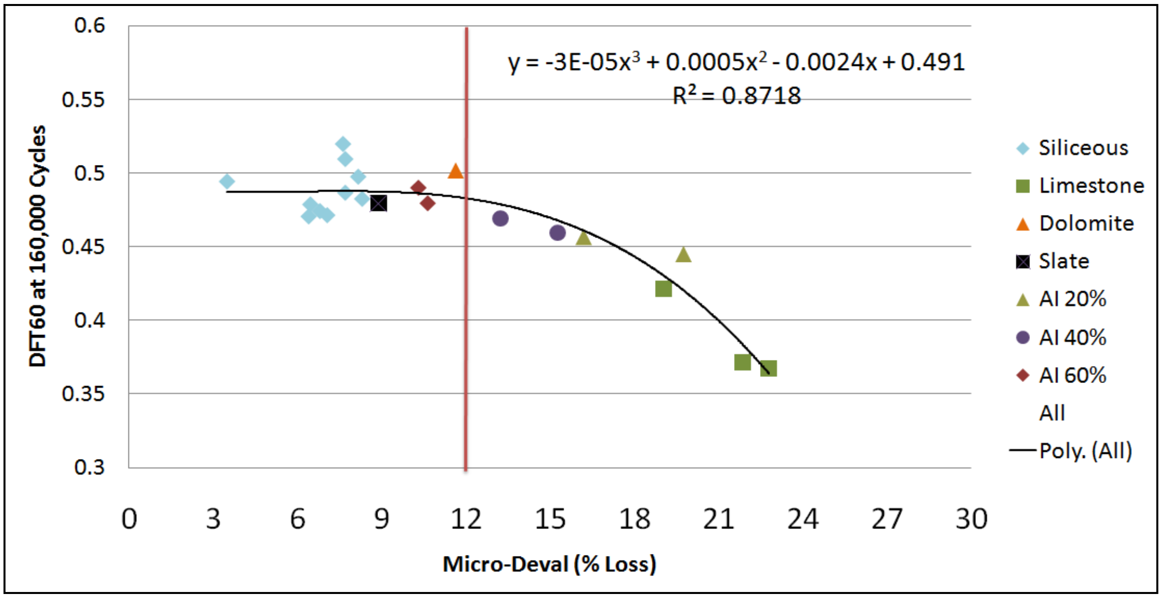

Figure 11.6: DFT60 vs. Micro-Deval (polynomial function)

The relation between DFT60 at 160,000 cycles and micro-Deval percent loss would be as follows:

$$DFT60 = (-3 \times 10^{-5} \times MD_{eq}^{3}) + (5 \times 10^{-4} \times MD_{eq}^{2}) + (2.4 \times 10^{-3} \times MD_{eq}) + 0.491 \quad \text{(eq. 11.1)}$$

MD_{eq} is the equivalent micro-Deval percent loss that could be computed using the following formula:

$$MD_{eq} = (\%\, aggregate\ 1\ \times MD_{aggregate\ 1}) + (\%\, aggregate\ 2\ \times MD_{aggregate\ 2}) \quad \text{(eq. 11.2)}$$

If a skid number value was to be computed in a similar way, it is preferable for that number to only account for micro-texture and not macro-texture. For this reason, it is better to relate polish resistance of fine aggregate to the skid resistance measured using a skid trailer with a ribbed tire. The formula provided by ASTM E 1960 includes the effect of macro-texture, and for this reason another formula should be used. Research done at University of North Florida and by NCAT on asphalt concrete found direct correlation between the DFT coefficient of friction at 60 km/hr and the locked-wheel skid trailer value measured at 40 mph using ribbed tires [Jackson, 2008; Heitzman, 2011]. Using the NCAT correlation, the $SN(40)_{smooth}$ value could be determined using the following equation:

$$SN(40)_{smooth} = (136 \times DFT60) - 19.4 \quad \text{(eq. 11.3)}$$

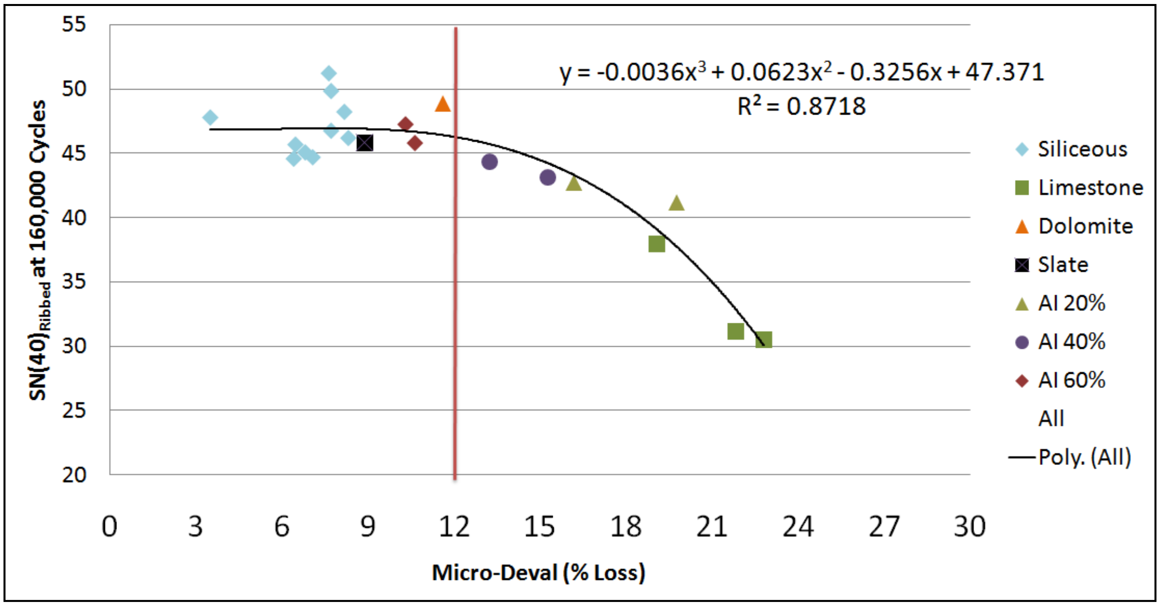

Figure 11.7: $SN(40)_{ribbed}$ vs. Micro-Deval (polynomial function)

Figure 11.7 shows the relation between $SN(40)_{smooth}$ computed using the NCAT correlation and the micro-Deval percent loss. The formula to estimate $SN(40)_{smooth}$ using the micro-Deval loss after 160,000 cycles is:

$$SN(40)_{smooth} = \left(-0.0036 \times MD_{eq}^3\right) + \left(0.0623 \times MD_{eq}^2\right) + \left(0.3256 \times 10^{-3} \times MDeq{+}47.4 \quad \text{(eq. 11.4)}\right.$$

Limited field testing was performed during this project, so it was hard to estimate what 160,000 TWPD cycles would correlate to in the field. However, for the field sections tested, high losses in skid resistance occurred for the sections containing 100% MFA on the truck lane within a year of casting that section. On the other hand, the blended sand section with the highest siliceous sand content seemed to have maintained its skid value within the last 5 years (comparing Table 9.6 to Figure 9.20). If this is compared to the values obtained from the laboratory testing, the following conclusions could be made:

- All mixtures cast on truck lanes experienced an initial drop in skid resistance after those sections are opened to traffic (within a year). The drop in skid resistance could be attributed to the loss of macro-texture contributing to skid resistance.
- Sections made with softer sands experienced a higher initial drop than the sections made with harder sands.
- Sections made with harder sands (or higher percentages of harder sands) were capable of maintaining skid resistance for a longer time after the initial drop in skid occurs.

Therefore, the equation for $SN(40)_{smooth}$ presented in this section could be used to estimate the initial drop in skid resistance based on the fine aggregates that are used in a concrete pavement. This equation, however, does not take into account traffic data, and should only be used to compare pavements made with different fine aggregates while assuming that those pavements are subject to the same traffic.

11.3 Recommendations for Accepting and Blending Fine Aggregates for PCC Pavements

The following method is recommended as an alternative method for accepting and blending aggregates for pavement concrete:

1. Test AIR (Tex-612-J) and micro-Deval (ASTM D 7428) for all fine aggregates. The AIR test will indicate the presence of carbonate aggregate, while the micro-Deval will measure the hardness of the fine aggregates.

2. Blend as indicated below based on aggregate test values.
   - If the fine aggregate has an AIR less than 60% and a micro-Deval percent loss higher than 12%, then that aggregate has to be blended with an aggregate that meets both those limits. (Range of acceptance; AIR > 60%; micro-Deval < 12%)
   - If the aggregate has an AIR less than 60% and a micro-Deval percent loss less than 12%, blend this aggregate with at least 40% of an aggregate that meets the AIR and micro-Deval limit.
   - Blend aggregates that do not meet either limit to result in an equivalent micro-Deval percent loss of 12% or less ($MD_{eq} < 12\% -$ computed using eq.11.2).
   - After determining the percentages of each aggregate needed to meet the 12% micro-Deval loss, ensure that at least 40% of the aggregate that meets AIR and M-D is used in the blend. If a lower percentage of aggregate meeting AIR and M-D is computed using the above formula, then use at least 40% of that aggregate.

If this method of blending is used instead of the current specifications, then more manufactured carbonate sand would be allowed in pavements if the manufactured sand itself was hard, or if it was blended with harder siliceous sands (hardness is evaluated by the micro-Deval test). Those recommendations also do not allow the use of 100% carbonate sand even if the sand used was a dolomite with a micro-Deval loss of less than 12%. The reason such recommendations were made was because no field sections containing 100% dolomites have been evaluated, so it is hard to recommend using such a sand in the field without first obtaining field performance data from a test section.

If blends of the siliceous and limestone aggregate tested during this research project were to be blended to meet a micro-Deval loss of less 12%, then the minimum

AIR that can be obtained from such blends would be greater than 40% (Figure 11.8). A field section containing a blend of aggregate with an AIR of 40% was evaluated (discussed in Chapter 9). That section seemed to have maintained good performance after 15 years of service.

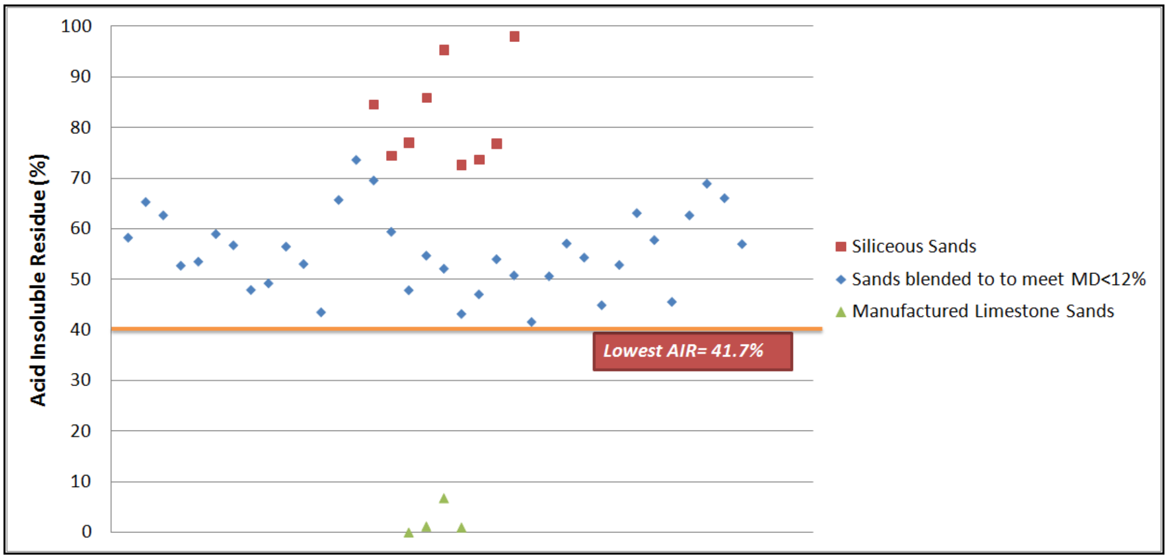

Figure 11.8: AIR Values for Blends of Aggregates Meeting the 12% Micro-Deval Limit

Chapter 12: Summary and Conclusions

12.1 SUMMARY

The goal of this project was to investigate the use of manufactured sands in pavement concrete. The reason this topic was investigated was because there is an increasing need to use more local materials such as manufactured fine aggregates that do not meet current specifications. The main concrete properties affected by the usage of manufactured fine aggregates are skid resistance, workability, and finishability. Skid resistance in PCC pavements is mainly a problem associated with the mineralogy of the sand. A large number of the available sources of manufactured sands are soft carbonate aggregates; those aggregates polish when used in PCC pavement. Workability and finishability are problems associated with the shape and grading of the fine aggregates used for making concrete. To obtain better workable and finishable concrete, the shape and grading of the aggregates has to be improved, or an optimized proportioning method that accounts for the poor shape and grading of those aggregates needs to be used. The research performed in this project investigated some of those issues. This section provides a summary of the different topics discussed in this dissertation.

12.1.1 Finding a Fine Aggregate Test that Predicts Skid Performance

The acid insoluble residue test currently used by the Texas Department of Transportation (TxDOT) and other state agencies is a surrogate test that measures the carbonate content of fine aggregates. The AIR does not directly measure the hardness of the aggregate. The use of the micro-Deval abrasion test for fine aggregates was explored. The time of the test was varied to investigate if better results could be obtained. Results showed that the 15-minute run time adopted by ASTM seems to give better results than the longer times attempted, because when the micro-Deval was run for longer periods of time more crushing of fine aggregates occurred.

Testing hardened mortar specimens in the micro-Deval might be a better way of evaluating polish resistant aggregates because it better simulates abrasion of fine aggregates in concrete. The problem encountered while testing mortar specimen using micro-Deval was that the abraded specimens had air voids that influenced the texture readings. Attempts to de-air the concrete worked, but fewer aggregates were exposed by the micro-Deval, so no consistent texture readings could be made on those specimens. The procedures to make and test the mortar specimen in the micro-Deval have not yet been optimized to the extent where reliable and repeatable results could be obtained; this is why the ASTM micro-Deval test was the only micro-Deval test that was compared to concrete results.

12.1.2 Evaluating the Shape of MFA Produced using Different Crushing Operations

To investigate if improvement in shape could be obtained by optimizing the crushing operation, two materials were sent to the Metso Mineral Research and Test Center (MRTC) in Milwaukee. The two materials MRTC were rocks obtained from the Lattimore Stringtown and Hanson Perch Hill aggregate pits. MRTC crushed each of those rocks using a Barmac B3000 VSI crusher at three different speeds. The Barmac B3000 was able to improve the shape of one of the aggregates (Lattimore Stringtown). The improvement in shape of the aggregate could be visually verified. The improvement in shape could not be quantified using AIMS; AIMS was not effective in evaluating the Lattimore Stringtown aggregates, mainly because AIMS is only capable of evaluating the 2D form and the angularity index of fine aggregates. AIMS failed to measure the flatness of the Lattimore Stringtown aggregate produced by Lattimore. Using the flow of mortar test described in ASTM C 1437 on re-graded sands was the best method used to indirectly evaluate the shape and texture of fine aggregate.

12.1.3 Proportioning Method for Pavement Concrete Containing Manufactured Fine Aggregate

The ICAR proportioning method for pavement concrete developed by McLeroy (2009) was first modified by replacing the visual shape and angularity rating scale by an AIMS function. The ICAR method was then used to proportion four sands; poor results were obtained for the sands with low microfine content because the method overestimated the amount of paste needed. To avoid overestimating the cement content it is recommended to only compute the minimum paste content and not to add any additional paste before trial batches are evaluated. The recommended procedure for proportioning pavement concrete could be summarized in the following steps:

1. Evaluating aggregate properties.
2. Plotting the conventional 0.45 power curve to determine the optimum gradation.
3. Performing a combined dry-rodded unit weight (DRUW) test on the selected proportions of aggregates to determine the paste content.
4. Performing trial batches to determine if additional paste is needed, or if increasing the admixture content is sufficient to obtain a concrete that meets slip-form concrete requirements.

It should also be noted that using the modified 0.45 power curve seemed to result in denser aggregate gradations, but it also resulted in aggregate proportions that caused shear slumps.

12.1.4 Developing a Laboratory Skid Test

The test developed for testing skid resistance of laboratory concrete specimens consisted of using a modified version of the NCAT polisher, a CTM, and DFT. The modifications made to the NCAT polisher consisted of replacing the pneumatic wheels with polyurethane wheels with durometer hardness equal to 85 and adding a vibration dampener between the gearbox and the turntable assembly.

The change in texture and friction was monitored over 160,000 polishing cycles using a CTM and DFT. Measurements were taken initially and after 5,000, 40,000, 100,000, and 160,000 polishing cycles. Compared to the results obtained using a CTM, the results obtained using the DFT better correlated with the expected performance of fine aggregate.

12.1.5 Evaluating the Skid Resistance of Pavements made with Sands that do not meet Specifications

Seven field sections in two different locations were evaluated for skid resistance using a CTM and DFT. Those sections were chosen because they were the only known sections that were made with materials that did not meet the acid insoluble residue (AIR) requirements. The three sections containing 100% carbonate MFA were constructed in 2008 as part of a TxDOT implementation project on the usage of MFA containing high microfine content in PCC pavements. Two of those sections that are located on the truck lane experienced a large drop in skid resistance a year after they were constructed; the skid value for those sections was even lower a year later.

Three sections constructed in the 1995 that contained blends of sands not meeting the TXDOT 60% AIR limit were also investigated. Those sections still maintained good skid resistance. The section with highest skid resistance was the section that contained the highest siliceous content.

12.1.6 Laboratory Concrete Tests

The CTM was found to be a good tool for differentiating between the different finishing techniques used. The DFT was found to be better than the CTM in evaluating the polish resistance of fine aggregates in pavement since it evaluates the micro-texture and not macro-texture.

The effect of changing fine aggregates on compressive strength, modulus of elasticity, drying shrinkage, and skid resistance were tested for concrete made with different fine aggregates was evaluated. The use of limestone manufactured sand at any replacement level in concrete did not significantly affect concrete compressive strength, modulus of elasticity, and drying shrinkage. Skid testing results showed that siliceous sands had higher friction values compared to limestone sands. The dolomite sand performed better than the other carbonate limestone sands. Results obtained also showed that blending a small quantity of siliceous sand with limestone sands considerably increased the skid resistance of concrete specimens.

The effect of changing mixtures proportions on skid resistance for mixtures containing MFA was investigated. Results showed that changing mixture proportions might have an effect on macro-texture, but it did not have any effect on the micro-texture. Thus, the performance of limestone manufactured fine aggregates were not improved by changing mixture proportions.

12.1.7 Correlating Aggregate Tests to Laboratory Concrete Tests

The results obtained from the concrete skid tests were compared to aggregate tests. Good correlation was found between the concrete laboratory test and the micro-Deval test. A formula that relates DFT60 and the skid number (SN) using a ribbed tire to micro-Deval percent loss was developed. Values obtained from this formula could be used to estimate laboratory concrete result by just testing the fine aggregate micro-Deval test. The equation for $SN(40)_{smooth}$ presented in 11.2 could be used to estimate the initial drop in skid resistance based on the fine aggregates that are used in a concrete pavement. This equation, however, does not take into account traffic data, and should only be used to compare pavements made with different fine aggregates while assuming that those pavements are subject to the same traffic.

Furthermore, recommendations were made for accepting aggregates or aggregate blends for pavement concrete. The recommendations should allow for higher usage of manufactured aggregate if hard siliceous or manufactured aggregates are used.

12.2 CONCLUSIONS

Good quality concrete can be produced using MFA if the aggregates are properly evaluated and the right proportions are used. Using 100% limestone sand is not recommended because it might cause workability and finishability related issues and will definitely cause loss of skid resistance. To obtain good skid performance using limestone MFA, MFA have to be blended with siliceous sands. For a given sand combination, the higher the siliceous sand content, the better the long-term skid performance. More MFA could be used in pavement concrete if those MFA are harder; for instance, a higher percentage of dolomite sand can be used in a blend compared to limestone sand blended with the same natural sand.

The workability and finishability of concrete made with manufactured sand could be improved if aggregates having better shape and grading are produced. If aggregates with good shape and grading are not available, then better proportioning methods (optimized) need to be used to minimize the paste content of concrete to produce a less costly and more durable concrete.

12.3 SIGNIFICANCE OF FINDINGS

The results obtained in this study will provide highway engineers with guidance on how to maximize the usage of their local sources of fine aggregate in PCC pavements. Under current specifications up to 40% manufactured carbonate fine aggregates could be used in PCC pavements. If the recommendations presented in Chapter 11 are adopted, up to 60% manufactured sand could be used in PCC pavement without significantly reducing skid resistance.

This study also demonstrated that pavement concrete mixtures containing MFA could be optimized by using relatively easy and simple methods. Using optimized concrete mixture will result in a cost reduction, a lower carbon footprint, and more durable concrete.

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Vita

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